HIGHWAY RESEARCH RECORD

Number 27

Highway and Interchange Capacity

6 Reports

Presented at the
42nd ANNUAL MEETING
January 7-11, 1963

HIGHWAY RESEARCH BOARD
of the
Division of Engineering and Industrial Research
National Academy of Sciences—
National Research Council
Washington, D. C.
1963

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30th Peak Hour Factor Trend

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•ONE of the problems of forecasting future traffic volumes is how the 30th peak hour ratio to the annual average daily traffic (AADT) reacts to an increase in the AADT. Experience has indicated that the design hour volume (DHV) rate of growth is not in the same ratio as the rate of growth of the AADT, although in the Highway Capacity Manual and AASHO Policy on Geometric Design Rural Highways, the following recommendations are made:

1. "The thirtieth highest hourly volume on a percentage basis changes very little from year to year....For example, if conditions indicate that a 20 percent rise in the annual average may be expected in 10 years, a similar 20 percent increase should be expected in the thirtieth highest hour; that is, if the facility is able to handle that much traffic." (1)
2. "Thus, the percentage of ADT for 30 HV from current traffic data on a given facility generally can be used with confidence in computing the 30 HV from an ADT volume determined for some future year. This consistency may not hold in instances where there is change in the use of the land area served by the highway. In such case, where the character and magnitude of future development can be foreseen, the relation of 30 HV to ADT may be based on experience with other highways serving areas with similar land-use characteristics." (2)

William Walker's findings showed that the earlier assumption of factor consistency was incorrect and that downward trends generally existed over the years (3).

The purpose of this study was to determine the relationship of the design hour factors to the AADT in the hope that developed trends would furnish a guide to predict future design hour factors.

DATA FOR STUDY

Sixty-nine counting stations that have been in operation in New Jersey for 10 years were selected as the basis of this study because they furnished the ADT, DHV and DHV factors. An analysis of these 69 stations indicated the following data (also see Tables 1 and 2):[1]

1. The Stations were located in all of New Jersey's 21 counties.
2. The ADT volumes ranged from 1,022 to 35,064.
3. The DHV's ranged from 200 to 4,280.
4. The DHV factors ranged from 7.6 to 71.5.
5. Sixteen locations are permanent counting stations and are counted 365 days per year.
6. Twenty-one locations are major counting stations and are counted one week out of every four.
7. Thirty-two locations are minor counting stations and are counted for one week out of every eight.
8. Thirty-six of the locations are located on rural highways and 33 locations are on urban highways.

Paper sponsored by Committee on Highway Capacity.

[1] All volumes quoted in this study are one-way volumes.

TABLE 1

GROUP	1951						1960					
	A. A.D.T.		D. H. V.		D. H. V. %		A. A.D.T.		D. H. V.		D. H. V. %	
	RANGE	AVERAGE	RANGE	AVERAGE	RANGE	AVERAGE	RANGE	AVERAGE	RANGE	AVERAGE	RANGE	AVERAGE
10 % LESS	29,235 3,887	13,140	2,430 350	1,310	8.3 9.3	9.0	25,550 4,929	15,450	2,480 460	1,470	8.4 12.1	9.8
10 % – 15 %	28,657 1,658	10,660	3,070 210	1,310	10.2 14.5	12.5	34,557 2,204	14,080	3,790 270	1,510	7.6 14.9	11.0
15 % – 20 %	16,425 1,161	4,400	2,920 200	760	15.2 18.8	17.1	22,115 1,489	6,150	3,100 240	820	10.9 16.4	13.4
20 % – 25 %	8,396 1,155	5,100	2,070 260	1,180	20.8 24.7	22.8	33,673 2,615	9,930	4,180 620	1,550	12.4 24.3	17.1
25 % – 30 %	8,010 1,022	4,820	2,170 280	890	25.0 29.0	26.9	12,466 1,689	6,580	2,130 290	1,350	17.1 23.8	20.7
30 % – 40 %	5,868 2,523	4,020	1,900 950	1,320	30.7 37.7	33.2	10,634 4,094	6,860	2,100 1,420	1,700	19.7 34.6	26.6
40 % – 50 %	3,007 1,426	2,000	1,490 580	910	41.0 49.6	45.0	6,321 2,223	4,270	1,870 800	1,330	29.6 35.8	32.7
50 % PLUS	2,265 1,420	1,980	1,230 850	1,070	50.4 60.0	54.9	2,967 1,379	2,130	1,180 640	870	31.7 56.8	42.8

TABLE 2

30th HOUR FACTOR GROUPS	STATIONS	PERMANENT CONTROL	MAJOR CONTROL	MINOR CONTROL	RURAL	URBAN	1951 A. D. T. GROUPS 1 – WAY					ROAD TYPE			
							0 to 2000	2000 to 3000	3000 to 5000	5000 to 10,000	10,000 PLUS	2 LANES	4 LANES	DUAL	VARIABLE
10 % LESS	7	0	6	1	2	5	0	1	1	1	4	3	2	2	0
10 % – 15 %	21	4	4	13	5	16	1	2	4	4	10	6	1	14	0
15 % – 20 %	15	5	2	8	12	3	3	4	3	4	1	10	2	2	1
20 % – 25 %	11	2	3	6	4	7	2	1	2	6	0	4	1	6	0
25 % – 30 %	5	3	1	1	4	1	1	0	2	2	0	2	0	3	0
30 % – 40 %	4	1	2	1	3	1	0	1	2	1	0	1	1	2	0
40 % – 50 %	3	0	2	1	3	0	2	0	1	0	0	2	0	1	0
50 % PLUS	3	1	1	1	3	0	1	2	0	0	0	3	0	0	0
TOTAL	69	16	21	32	36	33	10	11	15	18	15	31	7	30	1
	100%	23%	31%	46%	52%	48%	14%	16%	22%	26%	22%	45%	10%	44%	1%

GROUPING OF STATIONS

Since the design hour factor trends were the main purpose of this study the 69 locations were grouped according to their 1951 design hour factors as follows:

DHV Groups (%)	Stations in Group
10 or Less	7
10 - 15	21
15 - 20	15
20 - 25	11
25 - 30	5
30 - 40	4
40 - 50	3
50 - Plus	3

The average ADT, DHV, and DHV factors were calculated for each of the groups and the results and trends of the DHV factors are plotted in Figure 1.

Figure 2 shows the cumulative number of traffic counter locations having DHV factors of various values for 1951 and 1960. Figures 1 and 2 definitely show that the DHV factors have reduced over the 10-year period.

The yearly trend reduction rate for each group is as follows:

DHV Groups (%)	Yearly Change Trend (%)
10 or Less	+ 0.067
10 - 15	- 0.133
15 - 20	- 0.370
20 - 25	- 0.609
25 - 30	- 0.684
30 - 40	- 0.597
40 - 50	- 1.033
50 - Plus	- 1.190

From this illustration, it is clear that the reduction rate for the higher design hour factors is much greater than for the low design hour factors. These trend changes are similar to those of Walker (3).

The high design hour factor roads are generally those in sparsely populated areas. As the population and development along these roads increase, the DHV increases, but not in the same proportion as the ADT. This causes the design hour factors to decrease at their group reduction rate. When the design hour factor has reached a lower group rate, it then decreases at the new group rate. This cycle continues as the factor becomes smaller.

DETERMINATION OF REDUCTION RATE AND CURVE

To determine a trend rate of reduction for the DHV factors, a logarithmic straight-line trend was calculated based on the average DHV factors of all the groups. This trend gave a reduction rate of 2.3 percent compounded per year.

The exponential curve $Y = ab^x$, which represents a trend with a constant rate of decrease, was used to plot a curve. This curve seemed to fit the basic data reasonably; however, it approaches zero and the design hour factors can never be less than 4.2 percent of ADT.

To correct this, 4.2 was added to all points on the curve and to the basic data so that it would fit the curve (Fig. 3). This curve can be expressed by Eq. 1.

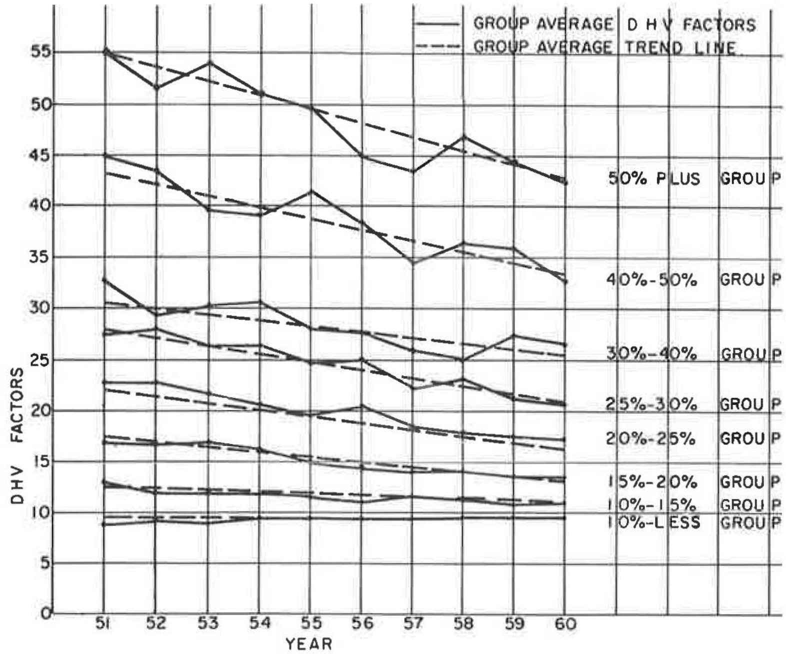


Figure 1. Group average DHV factor trends.

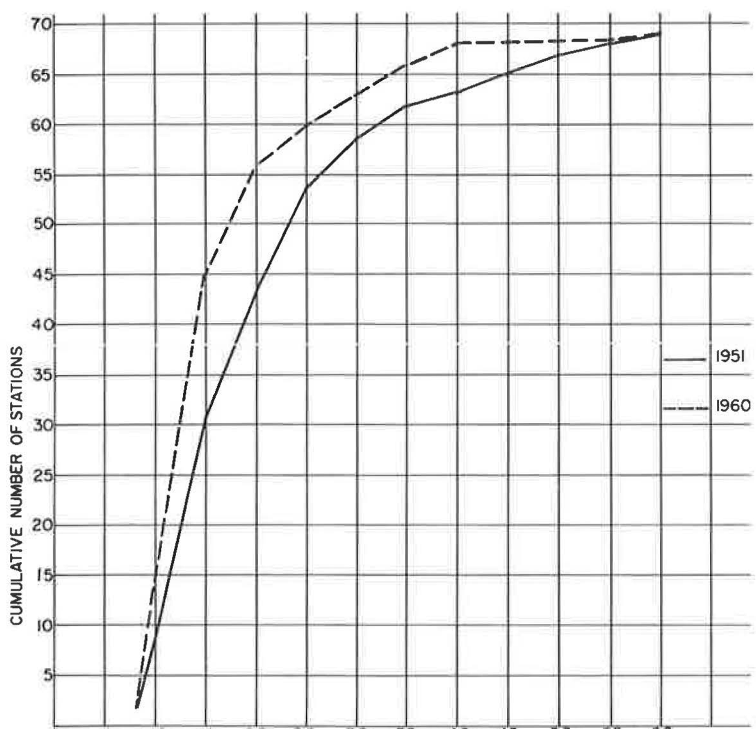


Figure 2. Cumulative number of traffic-counter stations having DHV's of various values.

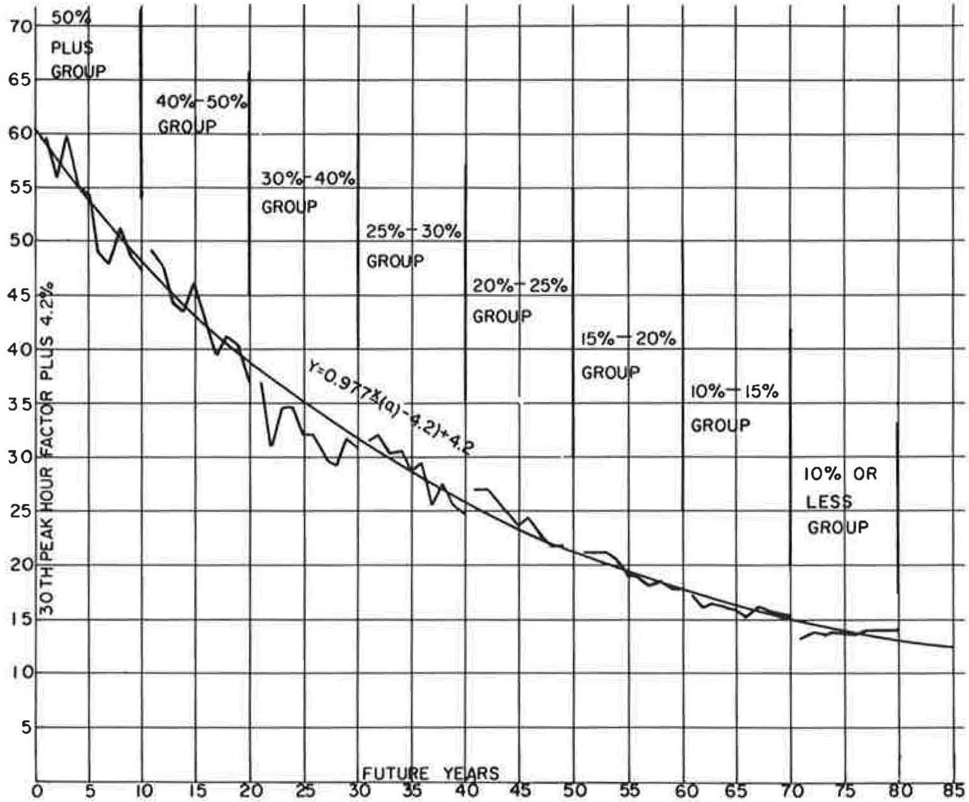


Figure 3. Trend curve and basic data.

TABLE 3

FUTURE DHV FACTORS (EQ. 1)

a	X = 5	X = 10	X = 15	X = 20	X = 25	X = 30	X = 50	X = 100
100	89.5	80.1	71.7	64.4	57.8	51.9	34.1	13.6
90	80.6	72.2	64.7	58.1	52.2	46.9	31.0	12.6
80	71.7	64.2	57.6	51.8	46.6	41.9	27.8	11.6
70	62.8	56.3	50.6	45.5	41.0	37.0	24.7	10.6
60	53.9	48.4	43.5	39.2	35.4	32.0	21.6	9.7
55	49.4	44.4	40.0	36.1	32.6	29.5	20.0	9.2
50	45.0	40.5	36.5	33.0	29.8	27.0	18.5	8.7
45	40.5	36.5	33.0	29.8	27.0	24.5	16.9	8.2
40	36.1	32.6	29.4	26.7	24.2	22.0	15.4	7.7
35	31.6	28.6	25.9	23.5	21.4	19.5	13.8	7.2
30	27.2	24.6	22.4	20.4	18.6	17.0	12.2	6.7
25	22.7	20.7	18.9	17.3	15.8	14.6	10.7	6.2
22.5	20.5	18.7	17.1	15.7	14.4	13.3	9.9	6.0
20	18.3	16.7	15.3	14.1	13.0	12.1	9.1	5.7
17.5	16.0	14.7	13.6	12.6	11.6	10.8	8.3	5.5
15	13.8	12.8	11.8	11.0	10.2	9.6	7.6	5.3
12.5	11.6	10.8	10.1	9.4	8.8	8.3	6.8	5.0
10	9.4	8.8	8.3	7.8	7.4	7.1	6.0	4.8
7.5	7.1	6.8	6.5	6.3	6.0	5.8	5.2	4.5
5	4.9	4.8	4.8	4.7	4.6	4.6	4.4	4.3

$b^1 = 0.977$ $b^{15} = 0.705$ $b^{30} = 0.498$

$b^5 = 0.890$ $b^{20} = 0.628$ $b^{50} = 0.312$

$b^{10} = 0.792$ $b^{25} = 0.559$ $b^{100} = 0.098$

APPLICATION OF CURVE AND REDUCTION RATE

Table 3 gives existing DHV factors and their future DHV factors for 5-yr intervals based on

$$Y = b^x (a - 4.2) + 4.2 \quad (1)$$

in which

Y = future DHV factor;
b = rate of reduction (constant 0.977 based on 2.3 percent compounded);
x = number of future years; and
a = existing DHV factor.

DHV FACTORS VS ADT

To determine the magnitude of the 30th peak hour factors for various volumes of traffic, the 69 counting locations were grouped as follows:

One-Way ADT Group	No. of Stations
2,000 or Less	10
2,000 - 3,000	11
3,000 - 5,000	15
5,000 - 10,000	18
10,000 - Plus	15

The average DHV factors of each group were calculated and the results are as follows (also see Fig. 4):

One-Way ADT Group	DHV (%)		Change	
	1951	1960	10-Year	Annual
2,000 or Less	28.3	21.1	- 7.2	- 0.7
2,000 - 3,000	25.8	18.8	- 7.0	- 0.7
3,000 - 5,000	21.5	16.8	- 4.7	- 0.5
5,000 - 10,000	19.7	15.2	- 4.5	- 0.5
10,000 - Plus	11.8	10.3	- 1.5	- 0.15

The following gives the DHV factor range for each ADT (one-way) group in 1951 and 1960 and the average DHV factor (also see Fig. 5):

One-Way ADT Group	DHV (%)			
	1951		1960	
	Range	Average	Range	Average
2,000 or Less	60.0 - 12.6	28.3	56.8 - 12.1	21.1
2,000 - 3,000	54.4 - 12.7	25.8	39.9 - 9.4	18.8
3,000 - 5,000	49.6 - 9.1	21.5	29.6 - 9.1	16.8
5,000 - 10,000	32.4 - 9.2	19.7	22.2 - 8.7	15.2
10,000 - Plus	17.8 - 8.3	11.8	14.0 - 7.6	10.3

Figures 4 and 5 show that the high design hour factors are on low-volume roads, and high-volume roads do not have high design hour factors. However, Figure 5 indicates that it is possible for both high and low volumes to have low design hour factors.

DHV FACTORS VS POPULATION CHANGES

Since it was felt that the population of the area might influence the DHV factor, an analysis was made at the 69 counting stations of the change in population and DHV factors. The population figures used were both those of the municipality and the county in which the counting station was located. Figures 6 and 7 indicate that an increase in population was accompanied by a decreased DHV factor. Samples of decreased population are too few to be significant (Table 4).

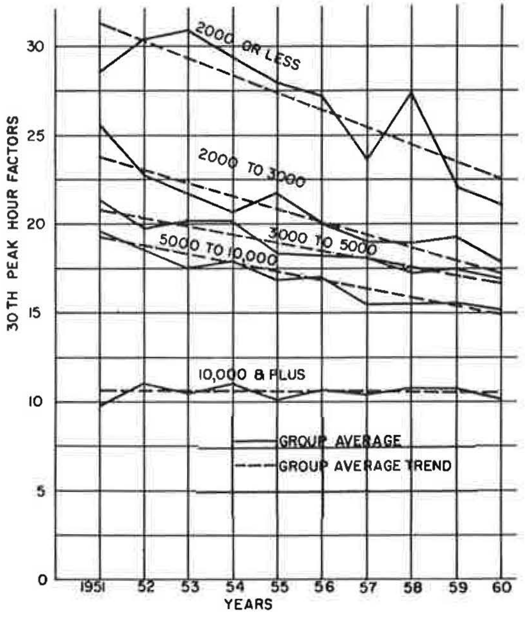


Figure 4. ADT group DHV factor trends.

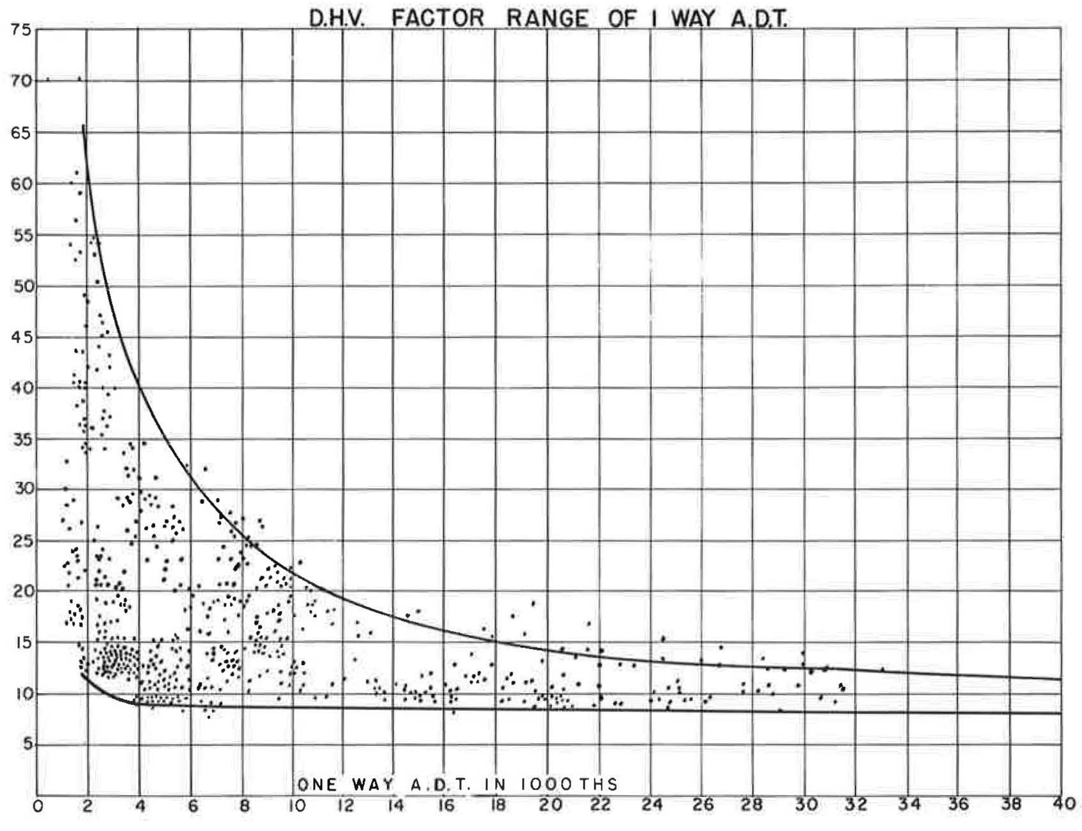


Figure 5. DHV factor range of one-way ADT.

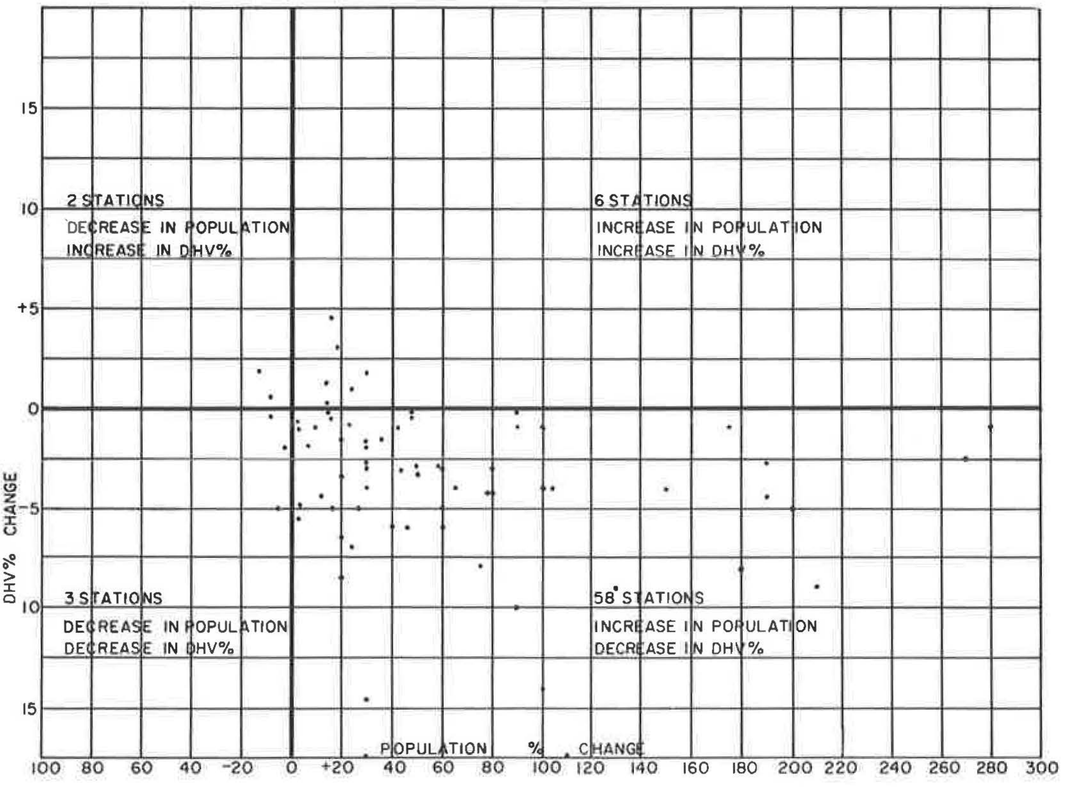


Figure 6. Ten-year DHV percent change vs ten-year population change of station municipality.

The average DHV factor and population of the station municipality for each DHV factor group was calculated for 1951 and 1960 (Fig. 8). The results also support the theory that a change in population of the area influences the DHV factors.

DHV FACTORS VS ROADWAY CAPACITY

To determine if the capacity of the roadway had any influence on the DHV factors, the satisfactory capacity and the tolerable capacity were determined for each of the 69 counting locations used in this study. (Satisfactory capacity is a level of service with 750 cars per hour one way on a 2-lane road and 1,950 cars per hour on 2 lanes of a 4-lane road; tolerable capacity is a level of service with 1,000 cars per hour on one way of a 2-lane road and 2,400 cars per hour on one way of a 4-lane road.) Having the yearly DHV for each location, the number of locations over or under these capacities was found for 1951 and 1960 (Table 5).

An analysis of the 69 locations indicated that the capacities had little influence on the DHV factors. Those over or under either of these capacities reacted like their respective group's DHV factors.

If the theory that capacities influenced the DHV factor is correct to any extent, then DHV factors would increase as the ADT decreased. This is not so.

Table 6 gives such locations. It is clear that the DHV factors were reduced as well as the ADT and DHV, indicating that capacity had little influence on the DHV factors.

Table 7 gives locations with increases in ADT and emphasizes the fact that the DHV does not increase in the same proportion as the ADT, which is the chief reason for the DHV factors decreasing.

The present recommended method of determining future DHV's is to apply the present DHV factors to the predicted future ADT.

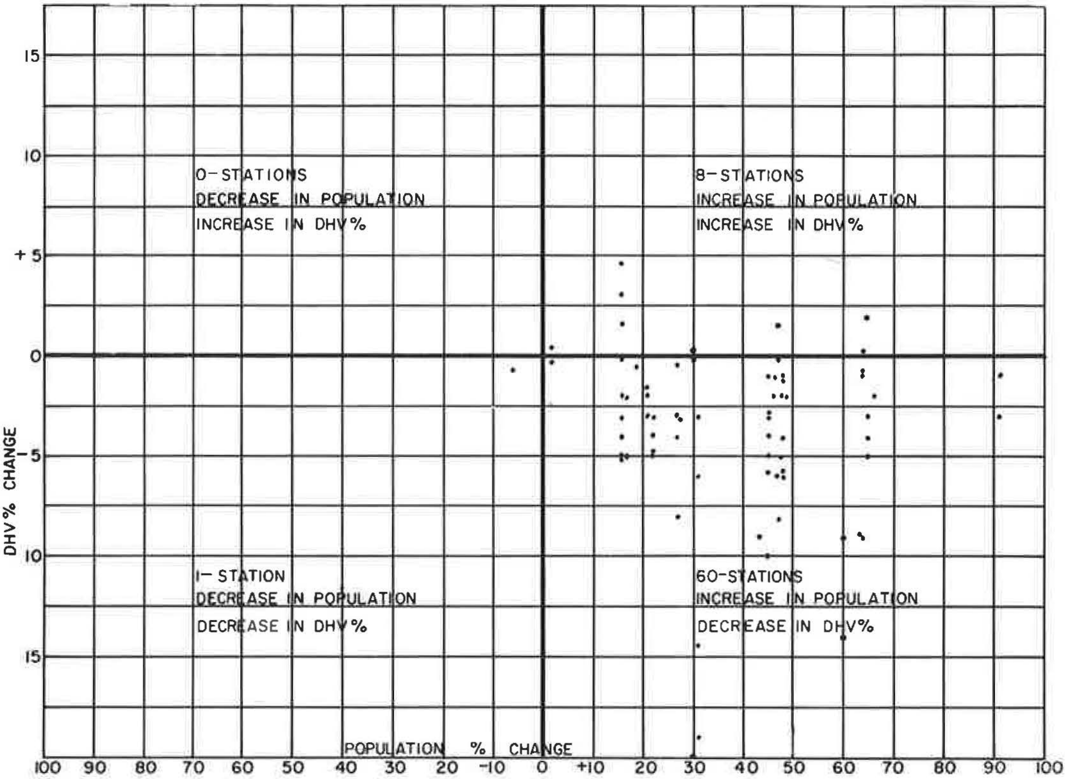


Figure 7. Ten-year station DHV percent change vs ten-year population change of station county.

TABLE 4

DHV FACTORS VS POPULATION CHANGES

DHV Factor Group (%)	Station (No.)	10-Yr Population Increase		10-Yr Population Decrease	
		DHV Factor Increase	DHV Factor Decrease	DHV Factor Increase	DHV Factor Decrease
10 - Less	7	3	2	2	0
10 - 15	21	2	17	0	2
15 - 20	15	0	15	0	0
20 - 25	11	1	9	0	1
25 - 30	5	0	5	0	0
30 - 40	4	0	4	0	0
40 - 50	3	0	3	0	0
50 - Plus	3	0	3	0	0
Total	69	6	58	2	3

Because this study indicated that the DHV factors reduce and do not remain constant, a comparison is made of the 1960 actual DHV's with the predicted 1960 DHV's by both methods at the 69 locations used in this study and at 19 locations that have been in operation only seven years.

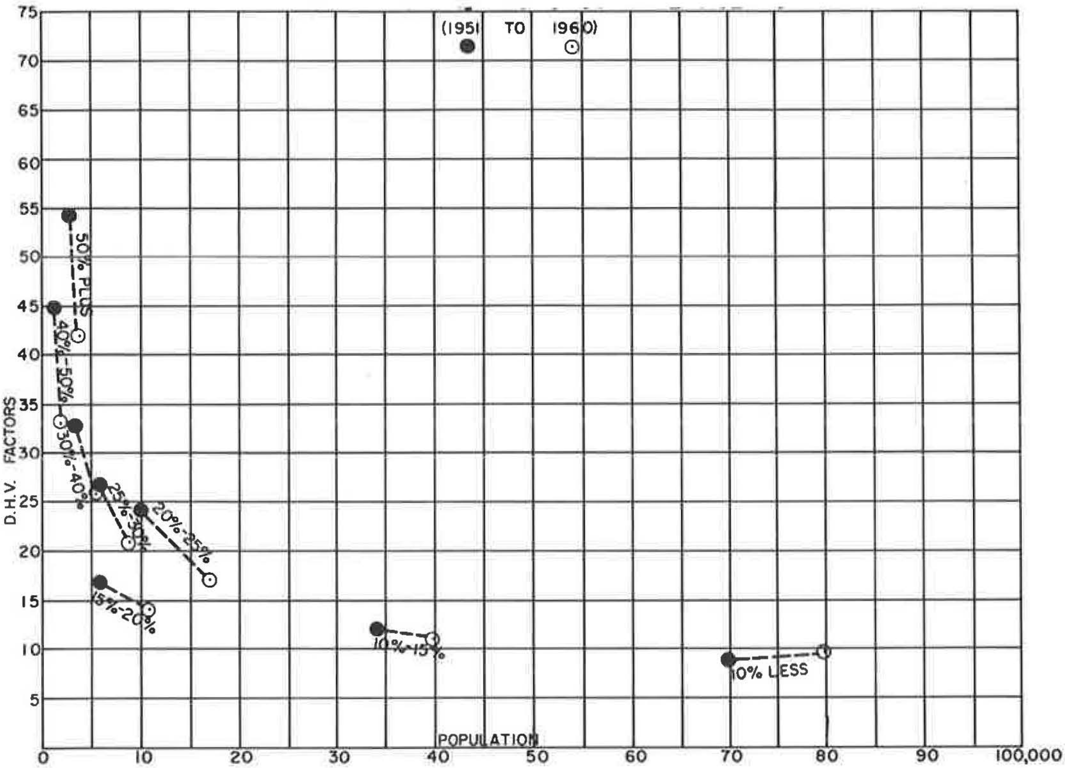


Figure 8. Group average DHV percent vs group average population.

TABLE 5

LOCATIONS OVER SATISFACTORY OR TOLERABLE CAPACITY

D. H. V. % GROUP	NO. OF STAS.	1951								1960							
		SATISFACTORY CAPACITY				TOLERABLE CAPACITY				SATISFACTORY CAPACITY				TOLERABLE CAPACITY			
		OVER	%	UNDER	%	OVER	%	UNDER	%	OVER	%	UNDER	%	OVER	%	UNDER	%
10 % LESS	7	2	29	5	71	1	14	6	86	3	43	4	57	1	14	6	86
10 % – 15 %	21	6	29	15	71	3	14	18	86	9	43	12	57	5	24	16	76
15 % – 20 %	15	2	13	13	87	2	13	13	87	2	13	13	87	1	7	14	93
20 % – 25 %	11	4	36	7	64	0	–	11	100	4	36	7	64	1	9	10	91
25 % – 30 %	5	2	40	3	60	1	20	4	80	2	40	3	60	1	20	4	80
30 % – 40 %	4	1	25	3	75	0	–	4	100	2	50	2	50	1	25	3	75
40 % – 50 %	3	0	0	3	100	0	–	3	100	1	33	2	67	0	–	3	100
50 % PLUS	3	3	100	0	–	2	67	1	33	2	67	1	33	1	33	2	67
TOTAL	69	20	29	49	71	9	13	60	87	25	36	44	64	11	16	58	84

TABLE 6

LOCATIONS WITH DECREASES IN ADT AND A DECREASE IN DHV FACTORS

US 1, New Brunswick, Middlesex County	1951	1952
ADT (1 way)	21,443	16,105
DHV factor	11.0	10.4
DHV (1 way)	2,360	1,670
US 130, Pennsauken, Camden County	1951	1952
ADT (1 way)	11,953	9,432
DHV factor	11.6	9.6
DHV (1 way)	1,390	910
US 130, Bordentown, Burlington County	1951	1952
ADT (1 way)	10,083	5,636
DHV factor	12.1	9.5
DHV (1 way)	1,220	540
US 130, E. Windsor, Mercer County	1951	1952
ADT (1 way)	8,670	4,823
DHV factor	12.4	10.1
DHV (1 way)	1,080	490
US 9, Pine Beach, Ocean County	1954	1955
ADT (1 way)	3,261	2,951
DHV factor	20.1	17.2
DHV (1 way)	660	510
US 9, Freehold, Monmouth County	1953	1954
ADT (1 way)	4,391	3,901
DHV factor	26.4	25.4
DHV (1 way)	1,160	990
N.J. 35, Middletown, Monmouth County	1954	1955
ADT (1 way)	9,429	9,307
DHV factor	17.4	14.5
DHV (1 way)	1,640	1,350
N.J. 35, Brielle, Monmouth County	1954	1955
ADT (1 way)	6,941	6,310
DHV factor	19.6	17.9
DHV (1 way)	1,360	1,130
N.J. 33 and 34, Wall Monmouth County	1954	1955
ADT (1 way)	7,084	6,202
DHV factor	27.4	24.5
DHV (1 way)	1,940	1,520

TABLE 7

LOCATIONS WITH INCREASES IN ADT AND A DECREASE IN DHV FACTORS

Garden State Parkway, Clark, Union County	1952	1960
ADT (1 way)	8,519	33,673
DHV factor	20.3	12.4
DHV (1 way)	1,730	4,180
N.J. 4, Paramus, Bergen County	1951	1960
ADT (1 way)	18,393	31,240
DHV factor	11.8	9.3
DHV (1 way)	2,170	2,910
US 22, Hillside, Union County	1950	1960
ADT (1 way)	20,580	31,511
DHV factor	12.0	10.1
DHV (1 way)	2,470	3,180
US 206, Bordentown, Burlington	1951	1960
ADT (1 way)	7,221	10,953
DHV factor	12.4	9.7
DHV (1 way)	900	1,060
US 46, Clifton, Passaic County	1951	1960
ADT (1 way)	12,424	21,976
DHV factor	13.4	10.8
DHV (1 way)	1,660	2,370
N.J. 3, Clifton Passaic County	1951	1960
ADT (1 way)	17,259	30,802
DHV factor	14.2	12.3
DHV (1 way)	2,450	3,790
N.J. 69, Hopewell, Mercer County	1951	1960
ADT (1 way)	3,064	4,231
DHV factor	15.2	10.9
DHV (1 way)	470	460
N.J. 18, Madison, Middlesex County	1951	1960
ADT (1 way)	3,409	7,362
DHV factor	22.1	13.6
DHV (1 way)	750	1,000
US 46, Ledgewood, Morris County	1951	1960
ADT (1 way)	8,010	12,466
DHV factor	27.1	17.1
DHV (1 way)	2,170	2,130
N.J. 23, Pequannock, Morris County	1951	1960
ADT (1 way)	5,868	10,634
DHV factor	32.4	19.7
DHV (1 way)	1,900	2,090
N.J. 73, Voorhees, Camden County	1951	1960
ADT (1 way)	3,007	6,321
DHV factor	49.6	29.6
DHV (1 way)	1,490	1,870

The following are the results of that comparison, showing at how many locations the predicted DHV's were closest to the actual DHV:

No. of Actual DHV Locations		Trend Method	No Change Method
1951 - 1960	69	50	19
1953 - 1960	19	13	6
Total	88	63	25

This clearly indicates that predicting the future DHV by use of the trend curve, developed by this study, is a more reliable method than the no change method.

CONCLUSIONS

1. The 30th peak hour factors generally decline as the AADT increases.
2. The reduction rate for high 30th peak hour factors is much greater than for low 30th peak hour factors.
3. Low population and sparsely developed areas, on the average, have a high 30th peak hour factor. Any marginal growth, such as housing developments, industry, or shopping centers, tends to lower the design hour factors.
4. Population changes in an area influence the DHV factors accordingly; an increase in population decreases the factors.
5. The capacity of a roadway has no great influence on the DHV factors or the rate of change. It is the increase in ADT due to the increase in the off hours that tends to reduce the DHV ratio to the ADT. Nevertheless, it is recognized that logically, when the potential 30th peak hour volume greatly exceeds the possible (absolute) capacity (such as may be experienced when the number of lanes are reduced for construction), the 30th peak hour factor may be reduced. But this is not supported by the study. This degree of over capacity condition has not been permitted to persist in New Jersey; therefore, this theory could not be tested.

REFERENCES

1. "Highway Capacity Manuel." U.S. Dept. of Commerce, Part VIII, 140-42 (1950).
2. "Policy on Geometric Design Rural Highways." AASHO, p. 56 (1954).
3. Walker, W.P., "Trends in the 30th-Hour Factor." HRB Bull. 167, 75-83 (1957).

Appendix

APPLICATION OF REDUCTION CURVE

To determine design hour factor for any future year when existing factor is known:

1. Locate the existing DHV factor on the curve (Fig. 9) and determine the year at this point.
2. This year point plus the number of future years for which the DHV factor is desired will locate the point on the curve where the future DHV factor can be read.

EXAMPLE

Existing DHV factor 50%.
What will it be in 20 yr?
1. Under 50% point on curve the year 15 is located.
2. 15 + 20 = 35
Above the year 35 the DHV factor of 33% is found.
Therefore, if the existing DHV factor is 50% in 20 yr hence it will be 33%.

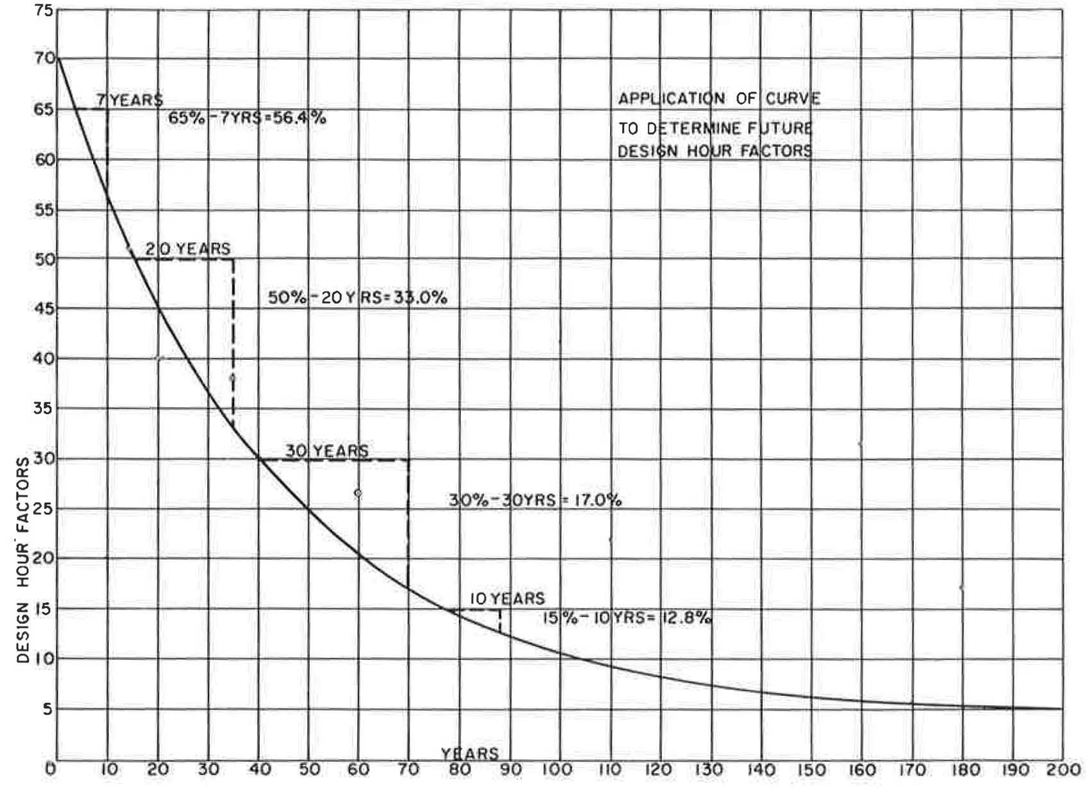


Figure 9. Modification of 2.3 percent reduction curve to approach 4.2 percent.

ESTIMATING THE 30TH PEAK HOUR

Four representative weekly counts of 168 hr each, one for each of the four seasons of the year, are selected as samples for each control counting station. The hourly volumes of traffic are then tabulated on a frequency table in an array arranged in convenient volume classes from the highest to the lowest volume. Since these four weeks of counts account for 672 hr, the total number of hours in each volume class is expanded by using a factor of 13. The total number of expanded hours then becomes 8, 736 hr, which is 24 hr short of the number of hours in a 365-day year.

The average volume for each class is divided by the AADT for the station and percentages are computed for use as ordinates on the final graph. These ordinates are plotted against abscissas derived from an array of the accumulated hours from the 13th to the 8, 736th hour.

The 30th peak hour is then estimated from the curve that was produced on the graph.

Traffic Operation at Two Interchanges in California

L. NEWMAN, Assistant Traffic Engineer, California Division of Highways

•THE CALIFORNIA Division of Highways has a continuing program of studying capacity and other operating features on various sections of completed freeways. These studies are made to better evaluate the extent and causes of problems that are occurring, thus permitting the possibility of finding the most economical solution, and learn more about basic traffic flow so that better and more reliable standards of design for future freeways can be developed.

The first location described is the Whipple Avenue Interchange on the Bayshore Freeway about 25 miles south of San Francisco, which is a cloverleaf type without collector roads. Many interchanges on California Freeways are of this type and therefore include a short weaving section 400 to 500 ft long.

Frequent observations of traffic conditions on these weaving sections indicate that problems occur even at relatively light volumes. Operation on the northbound lanes of the Bayshore Freeway at this location can be considered typical for volume ranges indicated. In general, drivers do not use as much of the weaving section as they could or should. Also, many of them stop or slow markedly prior to merging and actually wait for a gap long enough to be acceptable from a stopped position, rather than merge into available gaps in the freeway flow. Thus, the ramp vehicles are merging at a relatively slow speed compared to the freeway vehicles.

The second interchange area discussed is the westbound section of the Hollywood Freeway at the merge of the Franklin Avenue on-ramp.

Data from a larger study indicated that this location was a major bottleneck. Therefore, the basic data were examined in more detail to obtain traffic characteristics at the site, particularly flow characteristics of the Franklin on-ramp traffic which appeared to be the main factor in making this section a major bottleneck.

WHIPPLE AVENUE INTERCHANGE, BAYSHORE FREEWAY

The freeway proper at this location is not operating at capacity volumes even though the weaving volumes are at or near capacity (Fig. 1). However, at other cloverleaf interchanges (primarily the Stevens Creek Interchange on the San Jose-Los Gatos Freeway, where there are almost no trucks), despite the undesirable operation, capacity of the freeway proper is not necessarily reduced. (If capacity is defined as the maximum number of vehicles that can pass a point per unit time regardless of operating conditions.)

The study of the Whipple Avenue Interchange was undertaken for the following purposes:

1. To determine some of the basic characteristics of the operation of the weaving section.
2. To evaluate the effects of a method of striping a weaving section that is intended to encourage greater use of the auxiliary lane. (Essentially, it consists of a short solid stripe to guide the vehicles into a position parallel to the main freeway lanes and a dashed lane stripe to encourage the vehicles to stay in the auxiliary lane a longer period of time, thus permitting merging at a flatter angle and higher speed.)

Paper sponsored by Committee on Highway Capacity.

3. To test the feasibility of gathering data through time-lapse photography and its adaptability to other studies of this type.

The findings refer to the northbound lanes of the 6-lane Bayshore Freeway through the Whipple Avenue Interchange (Fig. 1). Observations were made during peak periods on two separate days. The first observations (6-21-61), referred to as the "before study," were made under striping conditions as shown in Figure 1. The second observations (9-13-61), referred to as the "after study," were made approximately one month after the section had been striped as shown in Figure 2. All data were obtained photographically. The camera was mounted on the butterfly directional sign just upstream of the interchange. Counting and other analysis were done in the office. Figures 3 and 4 are 9 sec of film for the before and after periods and are thumbnail descriptions of the study.

Peak Hour Volumes

Counts for the hour 7:10-8:10 AM were shown in Figure 5. This may not be the actual AM peak hour but it is very close. It was desired to photograph as late as possible for best light and also to insure obtaining some lower volume rates in addition to the highest rates.

The percent trucks and buses for the various movements are noted in parentheses. Of the 80 trucks during the before study approaching the interchange and going through (14 trucks were in lane 1 going to the off-ramp), 28 were in lane 1; 48 in lane 2 (including many large 5 axle trucks, etc.); and 4 in lane 3. During the after study of the 65 trucks going through, 32 were in lane 1; 30 in lane 2; and 3 in lane 3. This is a much higher percentage than is normal in lane 2 and is a result of relatively low traffic volumes in lane 2 plus a desire to avoid the possible conflicts in the weaving section.

Peak Flow Rates for Short Periods

The highest rate-of-flow for 5 min using the 130 ft radius on-ramp loop was 1,308 vph (during the after study). The highest 5 min rate-of-flow in the before study was 1,164 vph. The off-ramp flow rates during these 5 min periods were 588 vph and 744 vph, respectively.

Even during these periods there were several large gaps (10-16 sec) on the on-ramp indicating that possible capacity was not reached.

The highest weaving rate-of-flow for 5 min was about 2,100 vph and lasted for 2 consecutive 5-min periods (made up of 1,120 "on" and 980 "off" during the after study). For very short periods of about 30 sec, weaving rates as high as 3,100 vph were observed. This illustrates the very high volumes that can be maintained for a short period under ideal conditions including expert drivers. "Instantaneous" high-volume rates were more frequent, higher, and more sustained during the after study. These rates probably could not be maintained for 5 min (even if the demand existed) primarily because a significant number of drivers will stop and wait for gaps.

Operation of the Weaving Section

Speeds. —Speeds were calculated by measuring the distance traveled by a vehicle in a given length of time; 3 sec for lane 1 vehicles and 4 sec for on-ramp vehicles. The speeds are at the nose of the on-ramp and are not necessarily the speed during the actual merge.

During the lower volume periods (about 700 vph on the ramp) the average speed of the on-ramp vehicles is 27 to 30 mph. This speed is limited to a large extent by the 130-ft radius loop, although the speed was measured over a section essentially on straight alignment thus permitting some acceleration.

The speeds of lane 1 vehicles destined for the off-ramp at relatively low volume rates are about 40 mph. Speeds of the lane 1 vehicles going straight through were about 5 mph faster. This is about the same average speed that occurs at any exit ramp location when a substantial portion of the lane 1 traffic is destined for an off-ramp with a 130-ft radius at the exit nose.

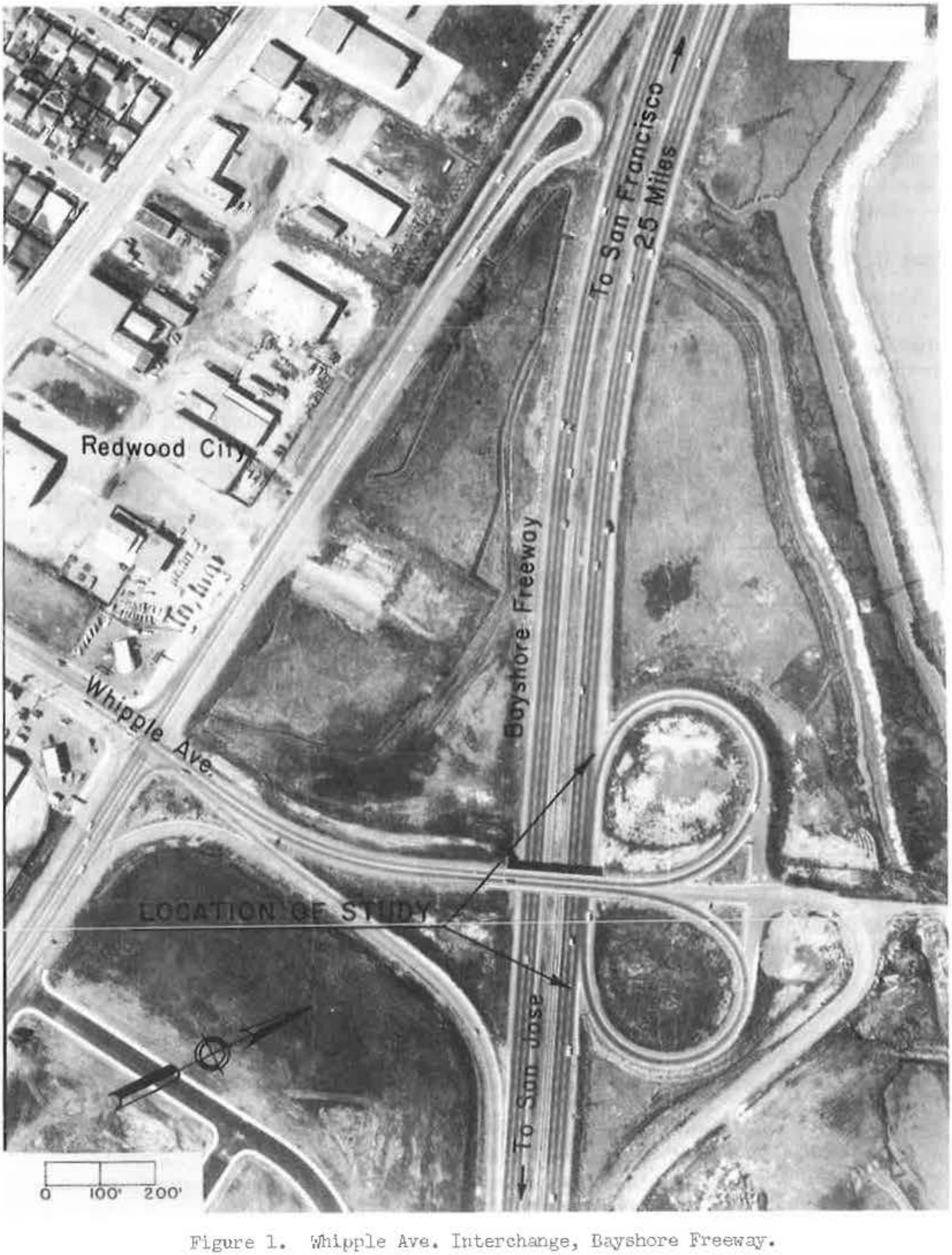


Figure 1. Whipple Ave. Interchange, Bayshore Freeway.

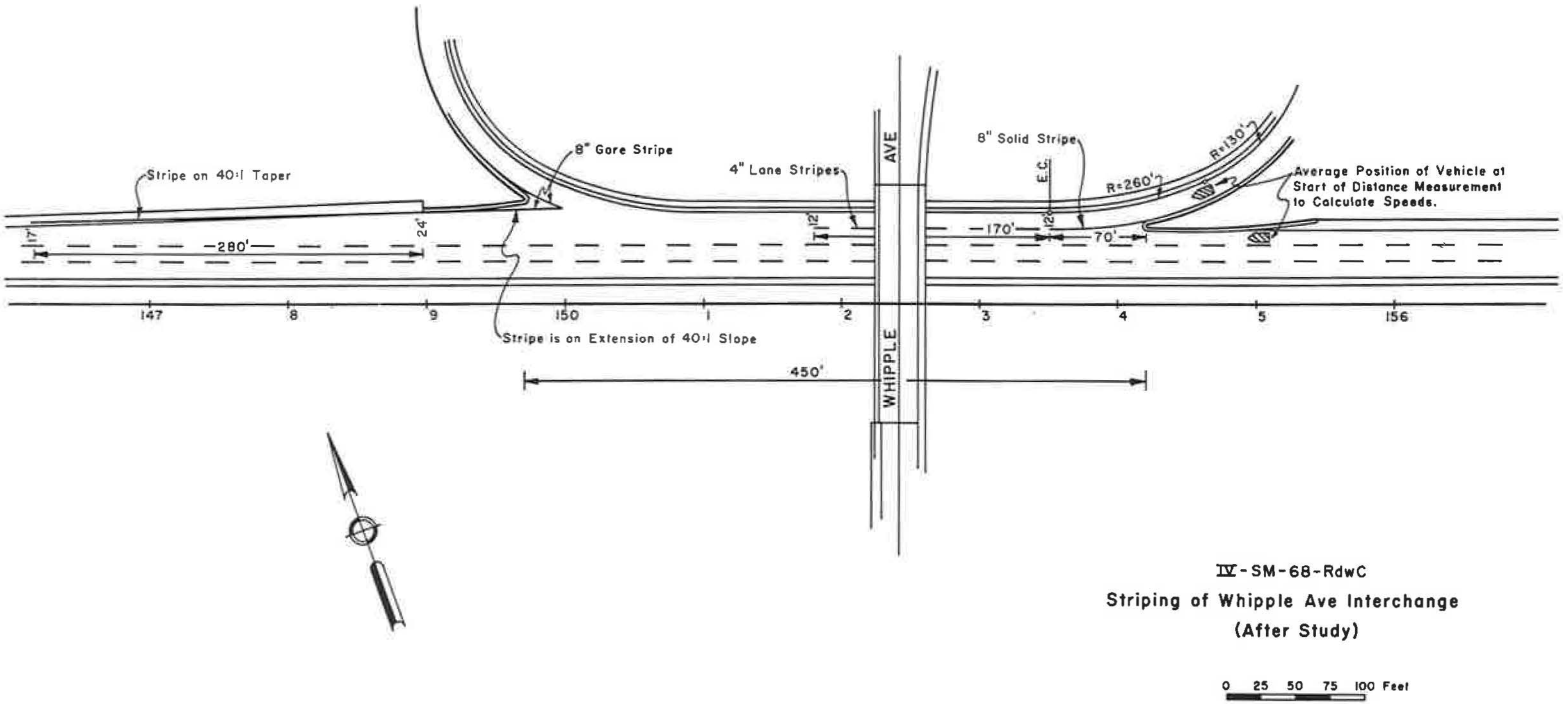


Figure 2. Striping of Whipple Ave. Interchange, after study.

Time - 0 sec

Series of 10 pictures 1 sec apart (elapsed time = 9 sec). During before period 7:55-8:00 rate for 5 min.

on = 815 vph
L_1 off = 515 vph
L_1 thru = 250 vph
L_2 = 985 vph
L_3 = 1,190 vph

Time - 1 sec

A vehicle (VW) reaches nose, and even though traffic is light, stops and waits for a gap. Note on-ramp vehicle ahead of VW.

Time - 2 sec

The vehicle (in shadow) ahead of the VW merges smoothly just ahead of a L_1 thru vehicle. Use of the weaving lane in this manner was much more frequent during the after study (Fig. 6).

Time - 3 sec

The dark vehicle in lane 1 in the foreground is going approximately 35 to 40 mph.

Time - 4 sec

The distance from the bottom of the picture to the nose is 115± ft. From the nose to the O. C. is 160± ft.

Time - 5 sec

This type of stoppage was more frequent during the before study (Table 1).

Time - 6 sec

The vehicle in lane 2 in the foreground is going 50 to 60 mph.

Time - 7 sec

The VW is still stopped and a second ramp vehicle is now arriving and is forced to stop.

Time - 8 sec

Three vehicles in lane 1 pass the nose during the 9 sec of elapsed time. This is a flow rate of only 1,200 vph.

Time - 9 sec

The VW has started to move and enters lane 1 after the station wagon passes.

Figure 3. Before study.

Time - 0 sec

Series of 10 pictures 1 sec apart (elapsed time = 9 sec). During after period 7:50-7:55 rate for 5 min.

on	= 730 vph
L_1 off	= 685 vph
L_1 thru	= 155 vph
L_2	= 1,020 vph
L_3	= 1,475 vph

Time - 1 sec

Sequence shows good use of weaving section at high instantaneous volumes.

Time - 2 sec

The light station wagon in lane 1 at nose has gone 125± ft in 2 sec equals 42± mph. Pickup in lane 2 equals 56± mph.

Time - 3 sec

The dark pickup on ramp uses most of the weaving lane and enters at a flat angle. The 2 on-ramp vehicles in foreground simultaneously merge into a 3.5± sec headway in lane 1.

Time - 4 sec

Time - 5 sec

Time - 6 sec

For a 6-sec period 3 lane 1 vehicles pass nose. Two ramp vehicles pass nose which equals rate of 1,800 vph weaving with 1,200; a combined rate of 3,000 vph.

Time - 7 sec

Time - 8 sec

Time - 9 sec

High flow rate in lane 3. Six vehicles in 9 sec equals 2,400 vph, 5 in first 6 sec equals 3,000 vph.

Figure 4. After study.

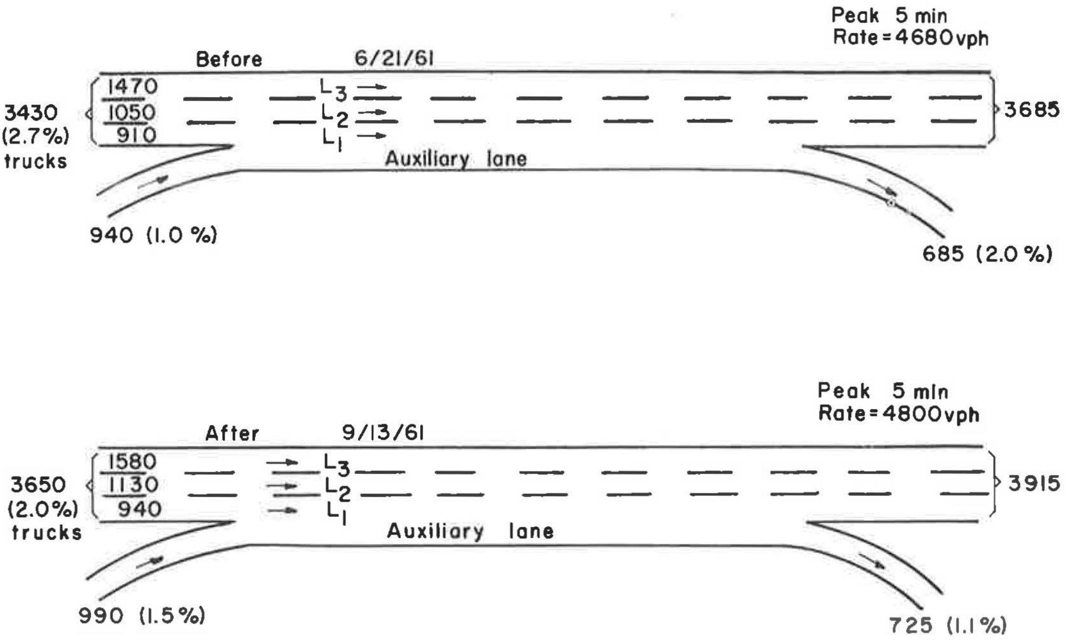


Figure 5. Peak hour traffic volume, Whipple Ave. Interchange study.

Speeds on lanes 2 and 3 were not obtained because operation on these lanes did not appear to be adversely affected by the weaving conflicts even during the most congested periods. The substantial number of trucks that were in the second lane had little or no effect on operation primarily because of the low volumes and high speed of the trucks on the level grade. Whether the same high percentage of trucks will use lane 2 when traffic demand for this lane increases is not known. If they do, then the operation and capacity of this lane could be affected.

Speeds would remain substantially at these levels until the weaving flow rates (for 5 min) exceeded 1,700 vph. When demand begins to exceed this rate, speeds of lane 1 and ramp vehicles are affected. Ramp average speeds at the highest volumes (2,100 vph weave) were 15 to 20 mph. Lane 1 speeds are not affected as much nor do they drop as quickly as ramp speeds. At the highest volumes lane 1 vehicles averaged about 30 mph with a substantial percentage at less than 25 mph.

Stoppages. —There were no queues or back-ups on the freeway and there were no freeway vehicles that came to a complete stop. But in two instances (once each during the before and after) several lane 1 vehicles slowed to 6 to 10 mph which to all intents and purposes is a stoppage. These occurred during the highest volume 5-min periods of both the before and after studies.

Numerous stops occurred on the ramp and long queues developed. During the before study there were 15 vehicles that came to a stop at the nose and waited for a gap (all but perhaps one were unnecessary). There were numerous other ramp vehicles that stopped but these were caused by stopped vehicles in front of them and not by any hesitancy to enter lane 1.

During the after study there were 10 vehicles that stopped prior to entering lane 1.

By contrast at a high standard on-ramp (Ashby Avenue southbound on the Eastshore Freeway) with a peak hour of 980 vehicles merging with 810 lane 1 vehicles, no ramp vehicles stopped at the nose and waited for a gap.

Use of Auxiliary Lane by On-ramp Vehicles. —During the before study about 89 percent of all on-ramp vehicles had entered lane 1 within 300 ft of the nose. Only about 65 percent were in lane 1 within 300 ft during the after study. There was little or no correlation between location of entry and approach speed, nor was there much correlation between on-ramp volumes and entry into lane 1 within 300 ft.

Vehicles entering lane 1 within 100 ft of the nose also were more frequent during the before study. In this case, there was a correlation between on-ramp volume and entry into lane 1 within 100 ft. As volume increases a greater proportion of ramp vehicles enter lane 1 within 100 ft of the nose.

Comparison of Before and After Studies. —Traffic demand during the after study was slightly higher although periods of capacity were reached during both the before and after study. The following points are pertinent:

1. Figure 6 shows a significantly greater number of on-ramp vehicles used the weaving lane more extensively—the primary purpose of the re-striping.

2. Approach speeds did not vary significantly (Fig. 7). Speeds when actually merging could not be obtained, but subjective observation showed the merging speed was higher, and smoother operation was obtained as a result of the vehicles staying in the auxiliary lane a longer period of time.

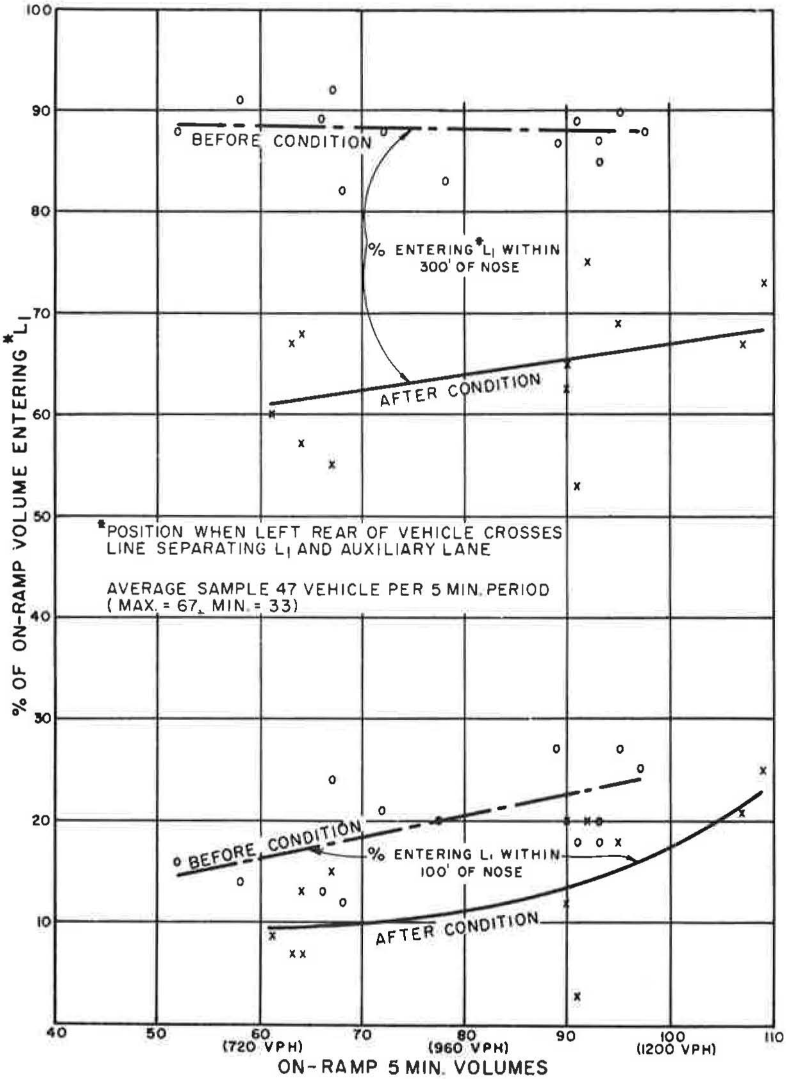


Figure 6. Use of auxiliary lane.

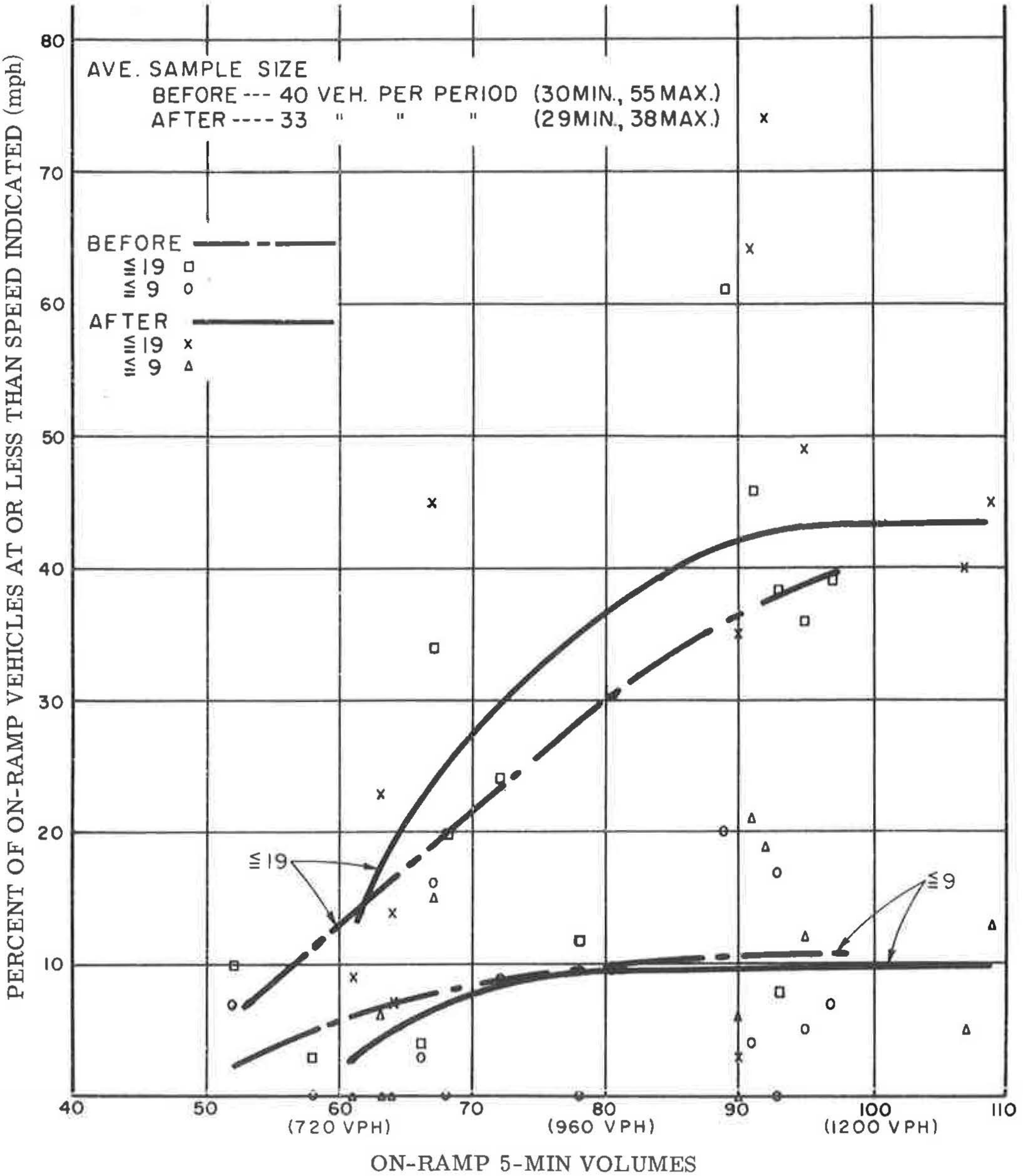


Figure 7. On-ramp vehicle speeds.

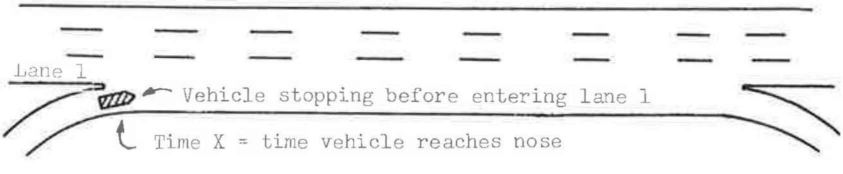
3. In spite of the higher volumes during the after period fewer vehicles stopped prior to merging into lane 1 (Table 1).

4. Indications are that actual capacity may be increased. The highest rate-of-flow for 5 min was as follows: weaving—after study = 2,088 vph, before study = 1,920 vph; on-ramp—after = 1,308 vph, before = 1,164 vph. Also during the after study there were instantantaneous periods (about 30 sec) which had higher rates-of-flow than were recorded during the before study (Table 2).

TABLE 1

RAMP VEHICLES STOPPING PRIOR TO MERGING[a]

5-Min Period[b]	Before			After		
	Veh/Hr[c]	Veh/Hr[d]	Sec[e]	Veh/Hr[c]	Veh/Hr[d]	Sec[e]
7:15-7:20	1,080	1,080	9	1,440	1,080	8
7:20-7:25	720	2,520	13	720	1,440	23
	1,800	1,800	9	1,080	1,440	10
7:25-7:30	1,080	1,080	8	360	1,080	11
	1,440	1,080	8	1,440	2,160	23
	2,520	1,440	18	--	--	--
7:35-7:40	1,080	1,080	8	--	--	--
	1,440	1,080	12	--	--	--
7:40-7:45	--	--	--	1,800	1,440	9
	--	--	--	1,080	1,080	7
7:45-7:50	1,440	1,440	12	1,080	1,800	23
	1,440	1,440	13	2,160	2,160	14
7:50-7:55	2,160	1,800	7	--	--	--
7:55-8:00	720	360	7	2,160	1,440	11
	1,080	1,080	9	--	--	--
	720	1,080	11	--	--	--
8:00-8:05	1,080	1,440	9	--	--	--



[a]Defined as any ramp vehicle which comes to a complete stop for at least 3 or 4 sec (or moves less than 5 ft ±) after reaching nose and before entering lane 1; stop is caused by hesitancy to merge and not influenced by preceding vehicle on ramp.
[b]Total no. of periods: before, 15; after, 10.
[c]Rate of flow in lane 1 for 10 sec preceding X.
[d]Rate of flow in lane 1 for 10 sec after X.
[e]Elapsed time from X when vehicle starts to merge.

Recommendations and Elaboration

The study shows that the re-striping as shown in Figure 2 resulted in significant improvement in the use of the weaving lane and in encouraging fewer vehicles to stop prior to merging.

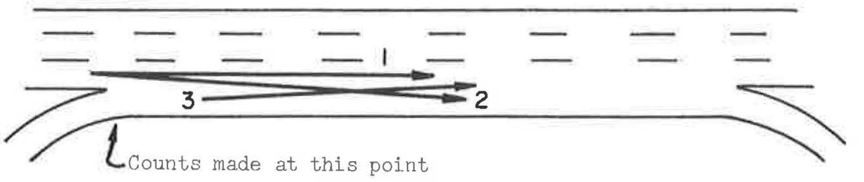
It is suggested that the striping shown in Figures 2 and 4 be adopted for all weaving sections in which the exit nose is offset 12 or more feet from the freeway, particularly those which have no differentiation in surfacing between the freeway and auxiliary lanes. The off-ramp vehicles were not affected and showed no hesitancy to cross either the solid stripe or dashed lane stripe to use the off-ramp.

This interchange works acceptably (that is, with little or no delay or conflict to lane 1 vehicles) as long as the weaving flow rate does not exceed 1,500 vph for 5 min. This corresponds to a peak hour weaving volume of 1,200 vehicles (using the same peaking characteristics as at this location). Possible capacity for 5 min appears to be about 2,200 vph weaving (with approximately equal amounts on and off) and with about 300 vph

TABLE 2

HIGHEST FLOW RATES FOR SHORT (≥ 14 SEC) PERIODS[a]

5-Min Period	Before					After				
	Veh/Hr				Time Period (sec)	Veh/Hr				Time Period (sec)
	Lane 1	Lane 2	Lane 3	Weave 2 and 3		Lane 1	Lane 2	Lane 3	Weave 2 and 3	
7:15-7:20	--	--	--	--	--	1,030	1,030	1,540	2,570	14
	--	--	--	--	--	400	1,600	1,200	2,800	18
7:20-7:25	290	1,580	1,150	2,730	25	--	--	--	--	--
7:25-7:30	160	980	1,640	2,620	22	510	1,800	1,800	3,600	14
	--	--	--	--	--	670	930	1,200	2,130	27
7:30-7:35	--	--	--	--	--	0	1,620	1,260	2,880	20
	--	--	--	--	--	140	1,300	1,150	2,450[b]	25
7:35-7:40	420	1,270	1,060	2,330	17	600	1,800	1,350	3,150[b]	24
	--	--	--	--	--	100	1,460	1,070	2,530	37
7:40-7:45	--	--	--	--	--	230	1,530	1,150	2,680	47
7:45-7:50	150	1,350	1,200	2,550	24	0	1,750	980	2,730	33



[a]In each case no stoppage occurred immediately after the end of the time period. The end of the time period was determined by a drop in demand (either in lane 1, the ramp, or both) and not by a drop in capacity due to stoppages. Several of these high periods occurred immediately after a vehicle which had stopped on the ramp (thus creating a backlog on the ramp) started to merge.

[b]The highest weaving rate for 5 min was 2,088 vph.

through in the right lane. At the rate traffic is increasing, these values probably will be tested before other highway improvements, expected to reduce ramp volumes at the interchange, are completed.

If a collector road existed, roughly the same congestion or conflicts would occur but would be moved onto the collector road. The weaving capacity would be the same. The primary benefit would be to the 200+ lane 1 through vehicles.

There would be no congestion whatsoever if this interchange were designed as a two-quadrant cloverleaf type; that is, if the off-ramp were taken off the freeway on a diamond type ramp thus leaving the freeway prior to the merge of the loop on-ramp traffic. As long as the crossroad is a normal surface street (with signals, etc.), capacity of the off-ramp and crossroad is seldom a problem, i.e., a diamond type off-ramp through the use of 2 abreast turns, etc., can supply just as much traffic as a single lane-loop and the surface street is just as capable of absorbing it.

Site and Study Method

Site Information.—The location of the study was the northbound lanes of the Bayshore Freeway as shown in Figure 1. The period of the study is the AM peak when much of the traffic is commuting to San Francisco (about 25 miles to the north) and other industrial areas farther north. There are employment and residential centers all along the

Bayshore Freeway that account for the high volumes using on- and off-ramps at approximately the same time. During the hour, one in every four cars in the northbound traffic stream just north of the interchange has come from Whipple Avenue.

The Bayshore Freeway at this point is 6 lanes on a level grade and is surfaced with asphalt concrete with no visible differentiation between the freeway lanes and the auxiliary lane.

The loops of the on- and off-ramps are on a 130-ft radius curve (measured along the right shoulder line).

Traffic operation on both days was normal. During the before study it was bright and warm. During the after it was overcast and cold (about 45 F) and many vehicles used lights for a portion of the hour. However, operation was considered normal.

Data Collection. —All data were collected photographically for the following reasons:

1. There was no set method for recording data or vehicle performance; but with a film record, the field situation could be repeated as often as necessary.
2. Manual data collection would have required so many men in the vicinity that normal traffic operation would possibly be affected.
3. Other methods of collecting data would not permit following paths of vehicles.

A 16-mm movie camera adapted for time-lapse photography was used. It was electrically driven from a portable generator.

A 100-ft roll of high-speed color film was used for each study ($12.50 per roll including processing). The rate of photography was 1 frame per second, and since there are 40 frames per foot of film, this allowed approximately 67 min of photography.

The camera was mounted on the directional sign preceding the on-ramp nose and operated from the ground, in preferance to using a conspicuous tower truck or other type of equipment. Once the study started, the equipment was left unattended, except for occasional checks. On another study of this type, the camera was mounted on an electrolier with good results.

A few traffic cones were placed to aid in determining distances to be used in analysis but other than that the appearance of the area was normal. Several of the district office personnel using this freeway did not notice that studies were being made.

Analysis. —All analysis was performed from the film and in the office, using a special projector. The film projection could be completely controlled by the operator for speed of projection or advanced or reversed frame by frame. There is no "flicker" regardless of how slow the film is run (accomplished by transporting the film between frames at a constant and very fast speed and varying the time the film is held still and projected to obtain the desired speed). Without these features it would be next to impossible to count or analyze from time-lapse photography.

Depending on density of traffic, counting could be done at varying speeds. At fairly light volumes counting could be done at projection speeds of 4 frames per second. All counts are at the nose of the on-ramp.

The projector is also equipped with a frame counter, obviating much of the need for a clock in the film.

Speeds and position of entry into lane 1 were determined by superimposing a grid on the film, and in the case of speed, recording the distance traveled over a given number of frames. When speeds of on-ramp vehicles were obtained, the vehicle's position when entry was made into lane 1 was also recorded (arbitrarily taken as the point when the left rear of the vehicle crosses the line separating lane 1 from the auxiliary lane).

The position at entry of about 50 percent of all on-ramp vehicles and speeds of from 30 to 50 percent of all lane 1 and ramp vehicles were determined.

Evaluation of Photographic Method. —The photographic method has the following advantages:

1. A permanent record of the field operation permits checking unusual looking results, and rerunning the peak hour as often as necessary to make subsidiary studies or evaluate unforeseen variables.
2. Vehicle paths can be followed and instantaneous type data obtained. For other than straight counting or spot speeds, manual observations cannot be adapted to collecting data for events that occur over a very short period of time. Other mechanical means

will not identify a vehicle. Several cameras can be used to cover an extended length of roadway simultaneously.

3. Although the equipment is relatively expensive, these are initial and fixed expenses. The cost of the film is negligible compared to transportation and overtime costs required of manual studies. If something unusual happens and the study data cannot be used, only the film and time of 1 or 2 men are lost. (For this study a platform truck was used to mount the camera and was kept in the vicinity during the study, but ordinarily this would not be necessary.)

4. Analysis of the film can be done in the office during regular working hours with no unusual manpower demands. No more man-hours would be required to record data from the film as would be required to record the same data manually as it actually happens.

There are some disadvantages, however. The method still requires a substantial man-hour effort to reduce the data depending on the amount and detail required. However, the amount of data reduction can be minimized, because detailed analysis need be performed only on portions of the period that are considered critical after a quick review of the entire film.

Also, detailed measurements can only be made for sections up to 400 ft, whereas 700 to 1,000 ft of roadway can be viewed subjectively. Speeds can be determined only to within 3 or 4 mph at the faster speeds. If lines were actually painted on the pavement, speeds could be measured more accurately. But generally, if more accurate speeds are required, other methods should be used or the camera would have to be a greater distance above the roadway.

Results

Table 3 summarizes the data by 5-min periods.

Volumes and Stoppages.—The total hour volume during the after study for off, lane 1 thru, and on was 1,931 vehicles or 4.3 percent greater than the 1,851 vehicles for these same movements during the before study. Most of this increase is primarily due to greater demand during the after study. But the maximum 5-min weaving volume rate during the after period of 2,088 vph is 8.8 percent greater than the before maxi-

TABLE 3

SUMMARY OF DATA, WHIPPLE AVE. INTERCHANGE STUDY

Period	Lane 3 (median)	Lane 2	Lane 1 Thru	Off	On	Weave On & Off	Ave. Speed (mph)			% On-Ramp Veh. Entering Lane 1 Within:					% On-Ramp Veh.		% of Lane 1 ≤ 24 MPH
							On	Off	Lane 1 thru	100 Ft	150 Ft	200 Ft	250 Ft	300 Ft	≤ 19 MPH	≤ 24 MPH	
(a) Before, NB TRAFFIC (Wed. AM, 6/21/61)																	
7:10-7:15	120	95	26	45	93	138	30	36	44	18	41	69	80	85	8	18	2
7:10-7:20	138	94	10	58	72	130	27	41	48	21	49	64	76	88	24	36	0
7:10-7:25	130	95	28	64	91	155	21	34	43	18	39	60	75	89	46	64	18
7:10-7:30	150	98	20	59	95	154	23	39	37	27	57	67	81	90	36	45	10
7:10-7:35	129	88	10	62	97	159	23	36	47	25	45	55	74	88	39	54	7
7:10-7:40	172	104	22	67	93	160	22	38	42	20	33	56	82	87	38	45	13
7:10-7:45	102	83	17	65	89	154	18	42	45	27	47	68	78	87	61	75	0
7:10-7:50	137	86	20	80	67	147	24	41	49	24	49	73	89	92	34	44	0
7:10-7:55	102	76	19	64	52	116	31	42	51	16	33	54	74	88	10	17	2
7:55-8:00	99	82	21	43	68	111	29	42	44	12	37	49	71	82	20	23	2
8:00-8:05	97	75	16	38	66	104	32	44	51	13	49	66	77	89	5	13	0
8:05-8:10	95	76	15	39	58	97	32	45	49	14	46	59	68	91	3	12	0
Total	1,471	1,052	226	684	941	1,625											
(b) After, NB TRAFFIC (Wed. AM, 9/13/61)																	
7:10-7:15	138	99	33	44	90	134	31	40	48	12	30	39	49	63	3	3	0
7:10-7:20	132	112	26	56	90	146	22	37	47	20	45	49	61	65	35	65	2
7:10-7:25	162	118	11	49	109	158	20	35	43	25	39	55	69	73	45	63	3
7:10-7:30	148	115	16	63	107	170	22	34	36	21	33	42	52	67	40	59	4
7:10-7:35	156	100	16	79	92	171	15	31	43	20	33	48	60	75	74	90	20
7:10-7:40	149	92	10	79	95	174	19	31	34	18	33	49	60	69	49	76	22
7:10-7:45	147	92	13	83	91	174	17	26	28	3	33	35	42	53	64	85	35
7:10-7:50	143	92	16	78	67	145	20	31	29	15	28	33	45	55	45	64	22
7:10-7:55	123	85	13	57	61	118	27	39	43	9	24	33	53	60	9	31	0
7:55-8:00	82	72	15	62	63	125	25	37	33	7	19	33	51	67	23	39	0
8:00-8:05	96	73	20	42	64	106	27	39	42	7	20	26	33	57	14	24	6
8:05-8:10	106	77	22	35	64	99	27	40	43	13	30	36	47	68	7	27	0
Total	1,582	1,127	211	727	993	1,720											

mum of 1,164 vph. These latter increases are believed to represent, to a certain extent, an actual increase in capacity.

Table 1 indicates the number of on-ramp vehicles which stopped prior to merging due to the ramp vehicle's own hesitancy to merge and not by any action of preceding ramp vehicle. Rates-of-flow are also given for the 10 sec prior to the time the vehicle which stops reaches the nose and 10 sec after it gets there, plus the time it takes the vehicle to merge after arriving.

Even though traffic volumes were higher during the after study, there were only 10 stoppages compared to 15 during the before study.

Table 2 gives the highest weaving rates observed during the before and after periods for short periods from 14 sec and up. It is not suggested that these rates could be maintained for long periods, primarily because some drivers will come along and wait for a gap. But the results show that during the after study they are more frequent, at higher rates, and over longer periods. This indicates a higher capacity with the after conditions, because, if more on-ramp vehicles use and stay in the auxiliary lane, as they do during the after study, they can merge simultaneously into single short gaps. If vehicles do not use the auxiliary lane they will merge one at a time, and a gap that could be used by two vehicles merging together will be used by only one vehicle.

Speeds.—Table 3 also gives average speeds for each of the 5-min periods, based on a 30 to 50 percent sample of speeds at the nose of the ramp; therefore, they are approach speeds rather than actual merging or weaving speeds. Unfortunately, speeds when actually entering lane 1 could not be obtained unless entry was made right at the nose of the on-ramp. (Vertical photography would be needed for accurate measurements.) Presumably the longer the ramp vehicle stays in the auxiliary lane, the faster it will be going when entry into lane 1 is made. Subjective observation bears this out.

The speed of a ramp vehicle waiting in queue to merge may be recorded as 0 to 10 mph even though it actually merges at a much faster speed.

To a large extent, therefore, speeds shown represent effects of the merge and depict a level of service. There were very few lane 1 through vehicles, so a single slow or fast vehicle could greatly affect the averages.

Normal on-ramp speeds of 27 to 31 mph and lane 1 speeds of 37 to 45 mph would be maintained until weaving volume rates exceeded approximately 1,500 to 1,700 vph, and no significant difference between the before and after period was noted. During the relatively free flow periods, the before shows higher average speeds although the differences are not considered significant because of recording error. The average lane 1 speeds for 5 min went below 30 mph only once and occurred during the highest volume period.

Figure 7 shows the percent of on-ramp vehicle speeds at or less than 19 mph and 9 mph. These represent less than normal on-ramp approach speeds and 9 mph or less, in effect, are stoppages. There is no significant difference between the before and after.

Use of Auxiliary Lane.—Figure 6 and Table 3 show the extent to which on-ramp vehicles use the auxiliary lane. Figure 6 shows the percent of on-ramp vehicles entering lane 1 within 300 ft and 100 ft of the nose of the on-ramp. As the on-ramp volume increases a greater percent will enter lane 1 within a shorter distance from the ramp nose.

FRANKLIN ON-RAMP TO THE HOLLYWOOD FREEWAY

The second interchange area discussed is the outbound lanes (west) of the Hollywood Freeway at high-volume conditions between the Franklin Avenue westbound on-ramp and Cahuenga Boulevard in Los Angeles. In addition to the general capacity characteristics such as volume, speed data, etc., detailed study was made of lane distribution and vehicle paths.

Capacity

Figures 8 and 9 show the geometric conditions affecting capacity of the section. Basically, it is a 3-lane section on a 1.2 percent uphill grade at an on-ramp with a

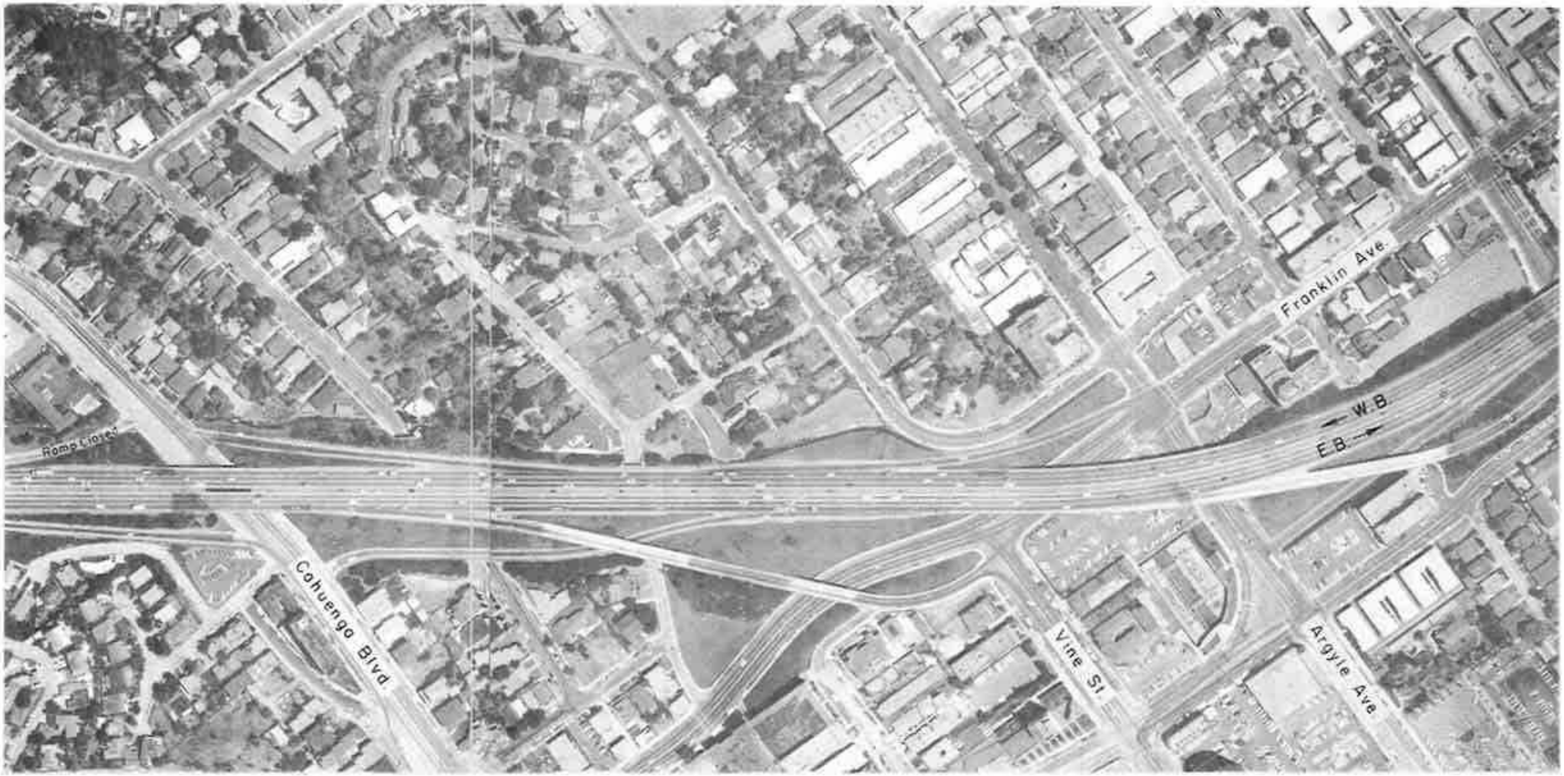


Figure 8. Hollywood Freeway study section.

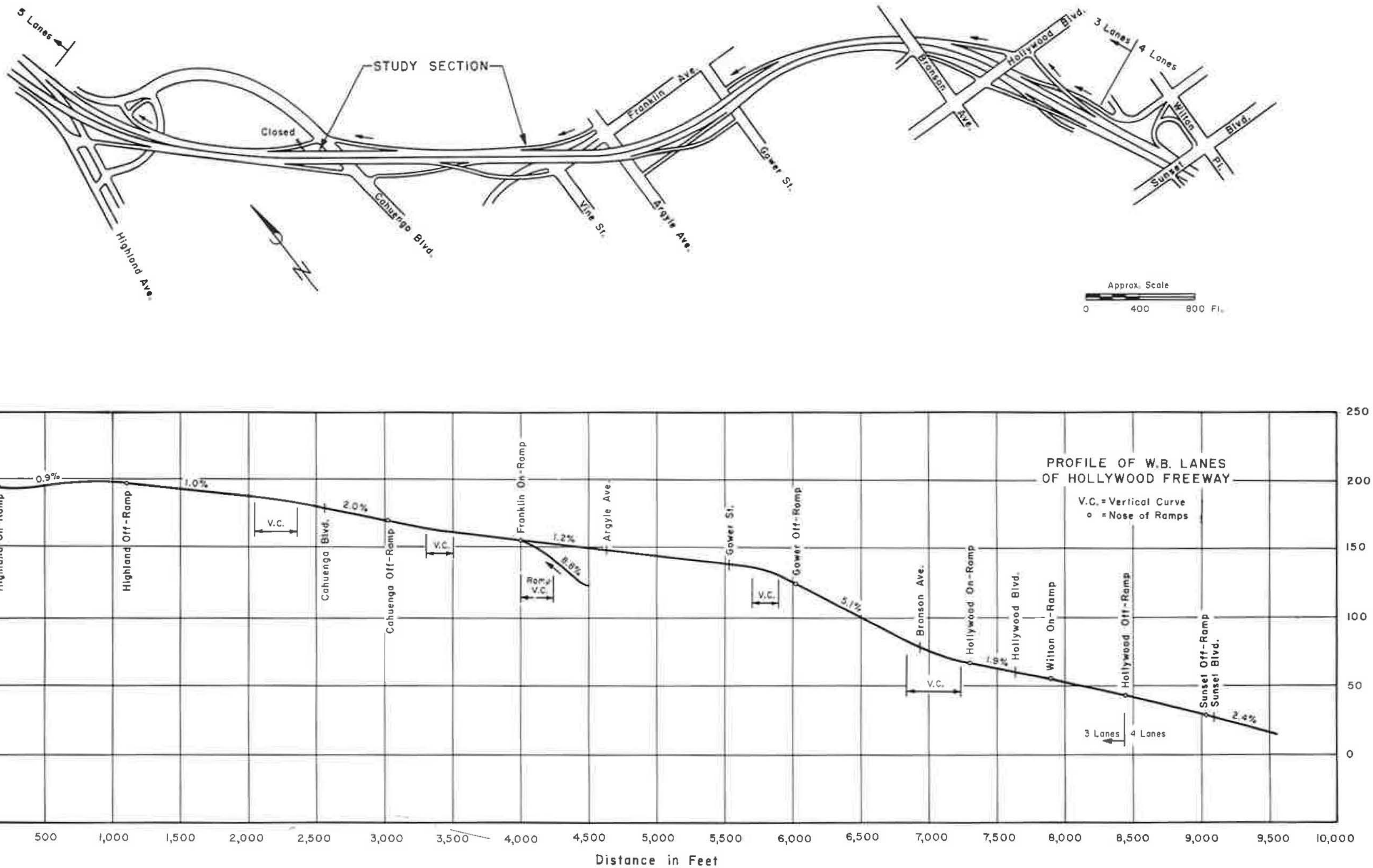


Figure 9. Hollywood Freeway between Highland Ave. and Sunset Blvd.

400-ft acceleration lane. This length (400 ft) is considered very short. The section is preceded by a 5.1 percent uphill grade about 1,200 ft long.

The sustained capacity of the section is about 5,500 vph including 500 to 800 vph on the ramp and 2.5 to 3.0 percent trucks. Five-min flow rates reach 5,700 vph fairly frequently, especially when on-ramp volume is low.

Effects of the upstream 5 percent grade on capacity of the study section are difficult to evaluate. It is doubtful if the capacity of the right lane is reduced in spite of the slower approach speeds, in as much as the combined volume of the ramp and right lane at the Franklin on-ramp is as high as could be expected considering the design of the ramp. It is possible, however, that capacity of the left lane has been slightly reduced because of slow speeds and the considerable friction on the grade. Capacity of the left lane at Cahuenga Boulevard is in the order of 2,000 to 2,100 vph for long periods instead of the 2,200 to 2,400 vph that has been obtained at other locations.

It is more likely that 2,400 vph is not obtained in the left lane because of the relatively large number of vehicles merging into the left lane in a short distance at slow speeds.

Capacity and Effects of the On-Ramp

As at other locations, ramp capacity is primarily determined by the number of vehicles in the adjacent freeway lane. The maximum short-term combined volume rate of the Franklin on-ramp and adjacent lane is about 2,000 vph, including 500 to 800 on the ramp. Two thousand vph is almost as much as can be obtained at any location with this ramp volume (2,200 vph is about maximum).

However, operation at high merging volumes, as reflected by lane 1 speeds, is much worse than occurs at ramps with better geometric design. At a combined merging rate of 1,800 vph, lane 1 speeds average 25 mph. At other locations with higher standard ramps, lane 1 speeds are seldom reduced below 35 mph.

Data developed in the report show that at a given total flow rate on the freeway (including ramp vehicles), the lower the ramp volume the better the freeway operation.

At an average total flow rate of 5,600 vph for 5 min, average freeway speeds were 20 mph or less for ten of the thirteen 5-min periods where the ramp volume rates were greater than 600 vph. When ramp volume rates were less than 600 vph with the same total flow rate of 5,600, there were no periods that had freeway speeds at 20 mph or less.

The implication is that 2 small volume ramps are better than one high-volume ramp, an observation borne out in studies at other locations. Thus, the common and expensive practice of combining two ramps, as in a two-quadrant cloverleaf interchange, so that they may join the freeway in a single merge is not necessarily the best solution. If adequate distance is available for a standard acceleration lane, two successive on-ramps should merge separately.

Lane Changing

In order to better understand lane distribution and factors limiting capacity, lane changing and vehicle paths were investigated. In summary, the following observations are made:

1. At capacity volume rates, 40 percent of the on-ramp traffic shifts out of the right lane within 1,600 ft downstream of the on-ramp nose. Most of the shifting occurs within the first 1,200 to 1,300 ft where pressure of high volume in the right lane is greatest. Included in the 40 percent are 7 to 10 percent that shift to the median lane in the same distance.

2. About 13 percent of the lane 1 traffic at the on-ramp nose shifts to the left within 1,600 ft. When comparing this with lane changing of ramp traffic that also must first merge into lane 1, it is apparent that traffic in the right lane is there for several reasons: (a) many vehicles are trucks that must stay in the right lane, (b) some vehicles are destined for nearby off-ramps, and (c) the drivers of others prefer to drive in the right lane or do not feel they can safely move to the left.

Traffic moving to the left lanes occasionally causes stoppages or shock waves in these lanes. Study of lane shifting shows that this and the resulting momentary stops do not necessarily reduce capacity of these lanes. It simply accounts for the low lane 1 volumes at points removed from ramps. Lane changing cannot be prevented, nor should it. But consideration of its effects should be noted and design features considered which will minimize adverse effects. These design efforts could include trying to reduce ramp volumes at one point as much as possible so that the number of vehicles changing lanes at once or in a short distance will be minimized; and placing special emphasis on good alignment and minimum grades at high-volume ramps so that high merging rates and resulting lane changing can be handled as well as possible.

Site and Data Collection

Maximum volume on the westbound 3-lane section of the Hollywood Freeway occurs between the nose of the Franklin on-ramp and Cahuenga Boulevard. This is the bottleneck in the outbound direction. The Franklin on-ramp is near the summit of a 5.1 percent grade about 1,200 ft long. The grade of the study section itself is between 1.2 and 2 percent. The ramp has a maximum grade of 8.8 percent approaching the freeway and only a 400-ft acceleration lane including taper. Almost no trucks use the ramp during peak periods.

This portion of the freeway was begun in 1951 and completed in 1953. With current design standards, the one significant difference in design would be the provision of an auxiliary lane between the on- and off-ramp.

It is fairly clear from traffic operation that the addition of the Franklin ramp traffic makes this section critical. It is not clear, however, how much the upstream grade affects capacity.

A given section of roadway has a certain capacity. Characteristics of traffic as well as the geometric design of the section can change the capacity of that section, e.g., a section of roadway on a 3 percent grade with 0 percent trucks has one capacity. If 5 percent of total traffic is trucks, the section has a different capacity. Furthermore, if there is an on-ramp at the section, the proportion of total traffic on the ramp affects the capacity as does the design of the ramp.

In the case of this section, perhaps the proximity of the 5.1 percent grade changes the characteristics of the approach traffic so that capacity of the study section is affected. (The capacity of the grade is definitely enough to load the study section at least from 5:00 to 5:30 PM; a change in the approach traffic characteristics means some change affecting its behavior when it reaches the study section.) Although this effect—if there is one—has not been determined, it is believed insignificant compared to other capacity determining factors at the study section such as percent grade and trucks, design and location of ramps, and amount of traffic using ramps. In other words, were the approach on a level grade, the capacity of the study section would still be essentially the same as indicated.

The peak hour volume using the upstream off-ramp to Gower Street is about 100 vehicles. Another off-ramp (Highland Avenue) with a peak hour volume of about 200 vehicles is 1,400 ft downstream of Cahuenga Boulevard. Two lanes are added to the section 1,200 ft further downstream at the left side of Highland Avenue on-ramp. The next off-ramp is Barham Boulevard, about 4,000 ft further downstream. The 5-lane section has enough capacity for the existing demand and traffic never backed up into the study section during the observed period.

Data were collected through time-lapse photography for two hours of traffic on two days, April 19 and 20, 1961. Data from two of the six cameras used in the larger study (operated simultaneously) were examined for this report. One camera covered the merging area at the Franklin on-ramp; other, the area between the nose of the Cahuenga off-ramp and Cahuenga Boulevard. Between the two sections, there was about 800 ft where detailed observations could not be made.

In studying the two locations simultaneously (to obtain lane changing, etc.), two projectors were used side by side so that vehicles could be visually identified at adjacent locations. Color film was used and the projectors were equipped for single frame and automatic operation.

TABLE 4

SUMMARY OF DATA, FRANKLIN ON-RAMP TO HOLLYWOOD FREEWAY

April 20, 1961 5-Min Period Starting	At Location 1[a]			At Location 2[b]			% On-Ramp Veh at Location 2 in:			On-Ramp Veh Identified (%)	Trucks & Buses at Location 1			
		Veh	Avg. Speed		Veh	Avg. Speed	L1	L2	L3		Buses	2-Axle	3 or More	Total
3:05	L1	61	38	L1	65	40	56	36	8	90	0	11	8	19
	L2	132	43	L2	126	44								
	L3	165	47	L3	153	42								
	Ramp	29												
				Total	344									
	Total	387			25	Cahuenga off-ramp								
3:10	L1	88	40	L1	77	38	17	52	31	80	1	12	20	33
	L2	131	35	L2	132	46								
	L3	163	42	L3	173	41								
	Ramp	36												
				Total	382									
	Total	418			25	Cahuenga off-ramp								
3:45	L1	77	35	L1	77	39	59	30	11	97	0	6	11	17
	L2	152	40	L2	146	42								
	L3	173	41	L3	181	39								
	Ramp	29												
				Total	404									
	Total	431			35	Cahuenga off-ramp								
3:50	L1	83	22	L1	99	30	53	37	10	78	0	5	7	12
	L2	147	23	L2	146	36								
	L3	157	26	L3	163	34								
	Ramp	40												
				Total	408									
	Total	427			26									
3:55	L1	110	26	L1	100	33	57	39	4	87	1	7	8	16
	L2	152	26	L2	155	38								
	L3	150	29	L3	158	35								
	Ramp	38												
				Total	413									
	Total	450			35									
4:00	L1	93	28	L1	101	38	57	30	13	89	2	9	9	20
	L2	145	23	L2	141	41								
	L3	156	28	L3	180	37								
	Ramp	29												
				Total	422									
	Total	423			31									
4:05	L1	89	22	L1	90	33	42	42	16	80	1	9	9	19
	L2	155	27	L2	150	38								
	L3	163	31	L3	178	36								
	Ramp	56												
				Total	418									
	Total	463			27									
4:10	L1	106	21	L1	101	33	54	39	7	87	1	14	11	26
	L2	146	27	L2	158	35								
	L3	161	27	L3	165	35								
	Ramp	38												
				Total	424									
	Total	451			21									
4:15	L1	106	19	L1	113	32	66	28	6	87	0	4	4	8
	L2	160	21	L2	150	30								
	L3	161	27	L3	166	28								
	Ramp	39												
				Total	429									
	Total	468			29	Cahuenga off-ramp								
4:20[c]	L1	57	37	L1	110	38	61	33	6	80	0	11	6	17
	L2	143	39	L2	153	37								
	L3	151	41	L3	168	34								
	Ramp	49												
				Total	431									
	Total	400			23									
4:25[c]	L1	65	38	L1	80	45	59	34	7	77	2	5	5	12
	L2	152	43	L2	150	45								
	L3	169	43	L3	178	38								
	Ramp	40												
				Total	408									
	Total	426			12									
4:30[c]	L1	77	42	L1	90	41	58	34	8	80	1	7	4	12
	L2	137	45	L2	143	45								
	L3	152	44	L3	157	41								
	Ramp	53												
				Total	390									
	Total	419			13									

TABLE 4

SUMMARY OF DATA, FRANKLIN ON-RAMP TO HOLLYWOOD FREEWAY (Cont'd.)

April 20, 1961 5-Min Period Starting	At Location 1[a]			At Location 2[b]			% On-Ramp Veh at Location 2 in:			On-Ramp Veh Identified (%)	Trucks & Buses at Location 1			
		Veh	Avg. Speed		Veh	Avg. Speed	L1	L2	L3		Buses	2-Axle	3 or More	Total
4:35	L1	92	30	L1	91	23	58	36	6	80	0	11	10	21
	L2	158	36	L2	164	33								
	L3	182	32	L3	184	34								
	Ramp	48												
				Total	439									
	Total	480			18									
4:40	L1	101	25	L1	112	34	71	26	3	77	0	4	7	11
	L2	152	27	L2	150	33								
	L3	162	28	L3	164	32								
	Ramp	47												
				Total	426									
	Total	462			15									
4:45	L1	96	29	L1	110	35	68	24	8	86	2	2	6	10
	L2	152	31	L2	155	35								
	L3	154	32	L3	167	35								
	Ramp	46												
				Total	432									
	Total	448			19									
4:50	L1	110	22	L1	121	32	64	29	7	90	1	4	4	9
	L2	156	22	L2	161	32								
	L3	170	29	L3	176	31								
	Ramp	67												
				Total	458									
	Total	503			20									
4:55	L1	119	16	L1	116	30	56	38	6	81	1	8	3	12
	L2	147	19	L2	158	32								
	L3	150	27	L3	164	30								
	Ramp	42												
				Total	438									
	Total	458			21	Cahuenga off-ramp								
5:00	L1	95	11	L1	118	29	70	28	2	92	2	5	8	15
	L2	154	18	L2	160	31								
	L3	156	25	L3	164	30								
	Ramp	64												
				Total	442									
	Total	469			19									
5:05	L1	93	12	L1	113	31	59	33	8	82	1	2	3	6
	L2	139	19	L2	155	30								
	L3	146	23	L3	159	31								
	Ramp	65												
				Total	427									
	Total	443			22									
5:10	L1	90	14	L1	115	33	66	27	7	82	3	2	5	10
	L2	144	18	L2	160	33								
	L3	142	24	L3	162	31								
	Ramp	76												
				Total	437									
	Total	452			15									
5:15	L1	83	12	L1	120	30	60	31	9	85	0	5	5	10
	L2	144	18	L2	160	31								
	L3	146	21	L3	161	32								
	Ramp	85												
				Total	441									
	Total	458			17									
5:20	L1	91	11	L1	119	29	61	30	9	85	2	2	6	10
	L2	136	17	L2	152	35								
	L3	155	22	L3	168	31								
	Ramp	68												
				Total	439									
	Total	450			14									
5:25	L1	93	17	L1	99	30	63	28	9	91	1	2	10	13
	L2	134	18	L2	146	32								
	L3	142	20	L3	158	33								
	Ramp	54												
				Total	403									
	Total	423			15									

[a]Location 1 at Franklin on-ramp nose.
[b]Location 2 at Cahuenga Blvd.
[c]Lower volumes and higher speeds are the result of a stalled vehicle restricting upstream traffic flow.

Peak Hour Volumes and Traffic Operation

Figure 10 summarizes counts for 4:30-5:30, shown in terms of hourly rates for the half-hours 4:30-5:00 and 5:00-5:30. This illustrates the differing pattern that occurs at the location (data by 5-min periods for one day are given in Table 4).

Sustained capacity of the section is about 5,500 vph. For 5-min periods, rates of about 5,700 vph are reached fairly frequently, especially when on-ramp volume is low. From 4:30 to 5:00 when the Franklin on-ramp was at a rate of about 600 vph, operation was considerably smoother than from 5:00 to 5:30 when the Franklin ramp volume was 800 vph even though the total freeway volume rates were the same.

Trucks and buses were about 2.7 percent of total volume from 4:30 to 5:00 and 2.4 percent from 5:00 to 5:30.

Average speed for all lanes at Cahuenga Boulevard on both days was 34 mph from 4:30 to 5:00 and 32 mph from 5:00 to 5:30. Flow at Cahuenga Boulevard was smooth throughout and no stoppages developed. This is evidence, even though speed was low, that traffic was leaving the bottleneck area.

At the Franklin ramp, operation was very poor, particularly after 5:00 when Franklin ramp traffic (from Hollywood's business center) increases. There were numerous stoppages on the ramp and right lane and several in the median lane as ramp and other traffic, under pressure of right lane congestion, moved to that lane downstream of the merge. Average speed for freeway traffic (excluding Franklin ramp vehicles) from 5:00 to 5:30 at this point on both days was less than 20 mph.

From 4:30 to 5:00 average freeway speed at this point was about 25 mph. Despite the higher speed, the section was the capacity limitation during this half-hour as well as from 5:00 to 5:30, because there was a continual backlog of vehicles on the grade. Also, data from the larger study indicated the capacity of the upstream grade was more than the 4,900 vph rate (5,500 less 600) that did negotiate it from 4:30 to 5:00.

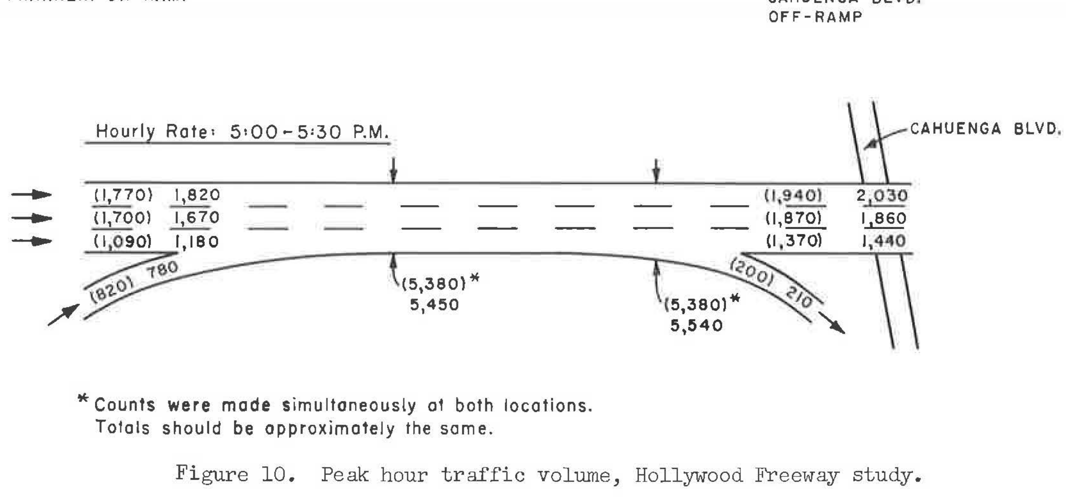


Figure 10. Peak hour traffic volume, Hollywood Freeway study.

Traffic Characteristics

Lane Distribution and Lane Changing. —Freeway traffic volume and distribution by lane at the Franklin on-ramp is different from that at Cahuenga Boulevard. There is considerably more traffic in the left lanes at the Cahuenga Boulevard location. Because the additional ramp traffic and significant shift of traffic to the left lanes is one of the limiting factors on capacity of the section (and the cause of occasional shock waves in the median lane at this point), an analysis of the ramp and lane 1 traffic was made.

Ramp Traffic. —Figure 11 shows the percentage of on-ramp vehicles that were still in lane 1 (right lane) about 1,600 ft downstream of the ramp nose related to total freeway volume. Even at capacity freeway volumes, about 40 percent of the ramp traffic has moved out of lane 1 with about 7 to 10 percent moving into lane 3.

Figure 12 shows the percentage still in lane 1 related to the volume of lane 2 and 3 traffic at Cahuenga Boulevard (including vehicles that moved over to fill these lanes).

Most of the lane changing takes place within the first 1,200 to 1,300 ft downstream of the ramp nose. This is the area of greatest pressure in the right lane which causes the rapid shift of traffic. Once Cahuenga Boulevard has been reached, a relatively normal distribution has been attained.

There is one 5-min period in which 83 percent of the ramp traffic had moved out of lane 1. Because this was such an unusually high percentage, it was checked in detail. Re-analysis indicated the percentage was correct and occurred because there was space available in the left lanes, and because there was an unusually large number of trucks during the period (33 or 8 percent of total traffic against a normal of less than 20 trucks per 5-min period; all were in the right lane).

There was no correlation between the percent of ramp vehicles shifting out of lane 1 and the number of ramp vehicles. In other words, the volume of on-ramp traffic is not a direct factor determining their lane changing.

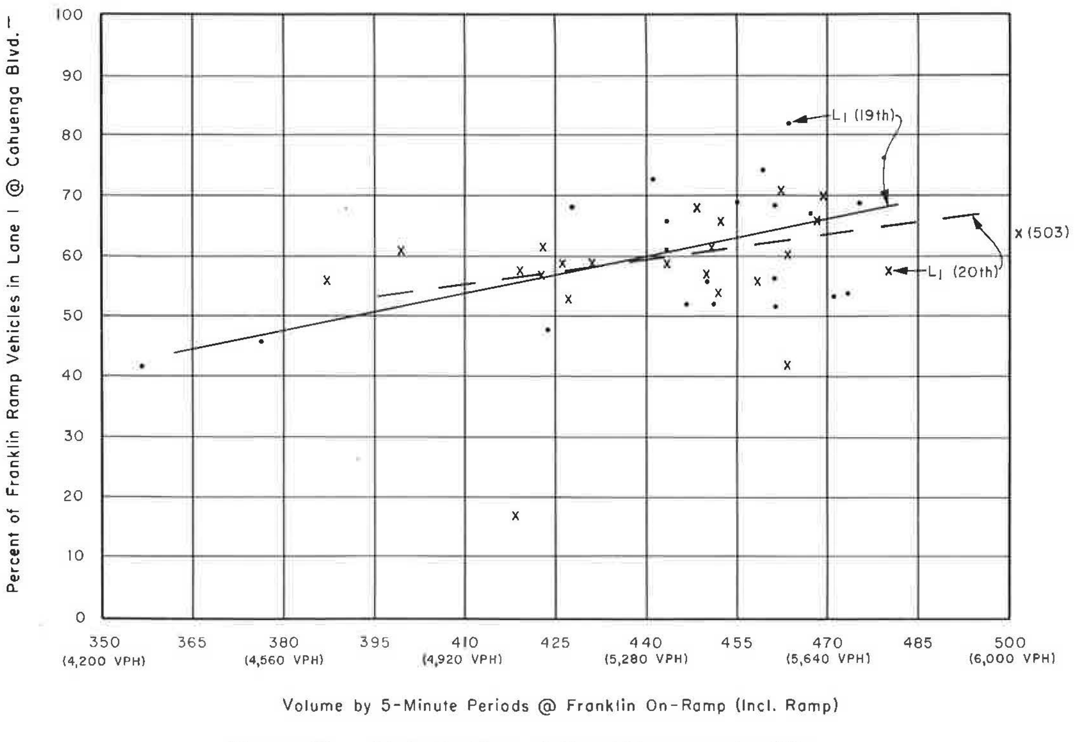


Figure 11. Distribution of Franklin ramp traffic.

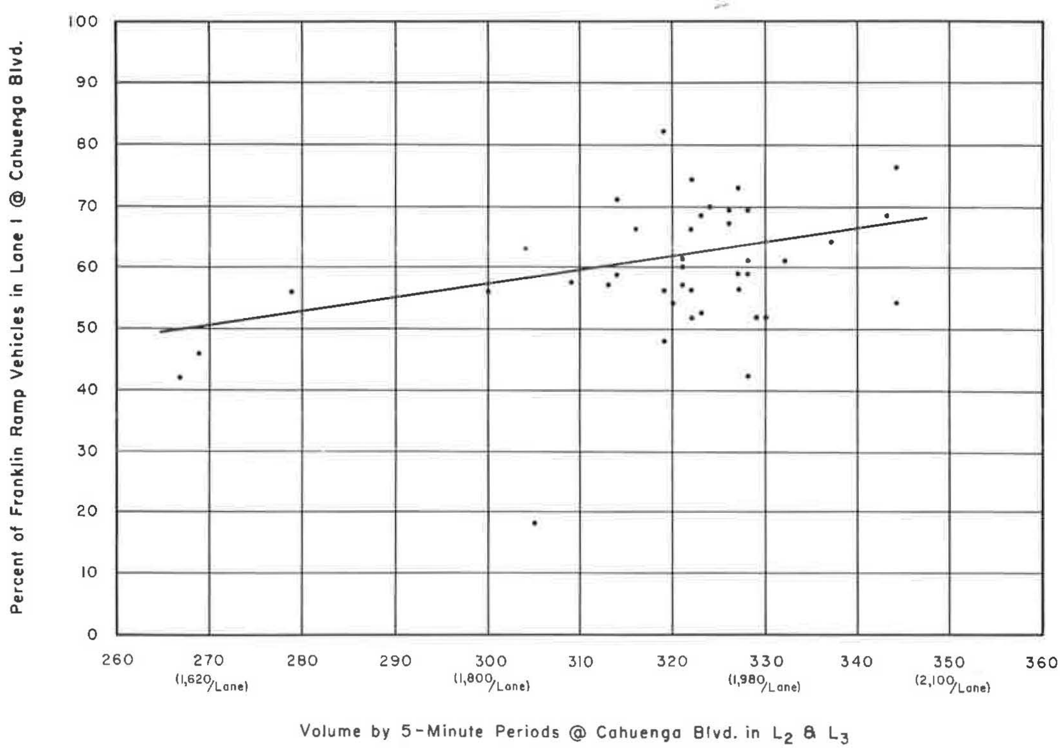


Figure 12. Distribution of Franklin ramp traffic.

A certain percentage of the on-ramp traffic is destined for the Highland Avenue off-ramp and generally it could be expected to stay in the right lane. Therefore, the percent of ramp traffic shifting out of lane 1 would be somewhat higher when only long distance ramp traffic is used as a base. For example, assuming a 5-min on-ramp volume of 50 vehicles, at Cahuenga Boulevard 25 may be observed in lanes 2 and 3, and 25 in lane 1. Therefore, 50 percent would be said to shift out of lane 1. If 5 of the on-ramp vehicles were destined for the Highland Avenue off-ramp, which is not unlikely, they probably should not be considered as part of the total. Then 25 out of 45 shifted out of lane 1 or 56 percent instead of the previously noted 50 percent. (Of the Franklin on-ramp vehicles, 2 to 5 percent exited at the adjacent Cahuenga Boulevard off-ramp; these vehicles were not included in determining the percent distribution at Cahuenga Boulevard.)

There is another factor also tending to make the actual number of ramp vehicles shifting to lanes 2 and 3 slightly greater than indicated (though it probably is not significant). For one reason or another, an average of 10 to 25 percent of the ramp vehicles could not be positively identified when they were at Cahuenga Boulevard. A greater percentage of the unidentified vehicles probably were in lane 2 and lane 3 than in the identified sample, because the location of the camera made identification of vehicles in lane 1 easier.

Right Lane Traffic.—Ramp vehicles quickly distribute to all freeway lanes, presumably because drivers believe they will be able to travel at higher speeds and encounter less congestion.

In the first 400 ft downstream of the on-ramp nose, (actual merging area) approximately 3 percent of the vehicles in lane 1 at the on-ramp moved to the left. In the next 1,200 ft, 10 percent more moved to the left—13 percent in 1,600 ft. Many of these lane 1 vehicles are probably ramp vehicles originating at the Hollywood and Sunset

Boulevard on-ramps 3,200 and 4,000 ft upstream of the Franklin on-ramp. The volume for the hour 4:30 to 5:30 on the two ramps together was 780 vehicles on the 19th and 710 on the 20th.

It was not possible to identify accurately the vehicles using the Cahuenga Boulevard off-ramp, but it was apparent that almost all of these vehicles were in the right lane at the Franklin on-ramp nose (1,000 ft upstream of the off-ramp). Therefore, about 15 percent of the lane 1 through vehicles moved overwithin 1,600 ft.

In summary, 15 percent of lane 1 through vehicles shift out of lane 1 within 1,600 ft, as compared with 40 percent of the ramp vehicles that must first merge into lane 1 using 400 of the 1,600 ft for this purpose.

It is obvious then that lane 1 traffic generally is there for a purpose and most will not shift over in spite of the congestion in this lane.

Effect of Franklin On-Ramp on Freeway Operation and Capacity.—The acceleration lane (including taper) is only about 400 ft long, considerably less than current standards. The ramp approach is on a 8.8 percent upgrade. However, almost no trucks or buses use the ramp. Another important factor is the freeway upgrade approaching the ramp which causes slower than normal lane 1 speeds even at low volumes.

Figure 13 plots average speeds of lane 1 at the on-ramp against total ramp and lane 1 volume. Maximum combined lane 1 and ramp volume is between 2,000 and 2,100 vph. The highest rate observed at other merging locations with comparable ramp volumes is about 2,200 vph. However, operation at high volumes as reflected by lane 1 speeds is much worse than observed at locations with higher standard ramps. For example, at a combined volume rate of 1,800 vph (including 500 to 800 vph on the ramp), average lane 1 speeds are about 25 mph. At other locations (i. e., Ashby Avenue on

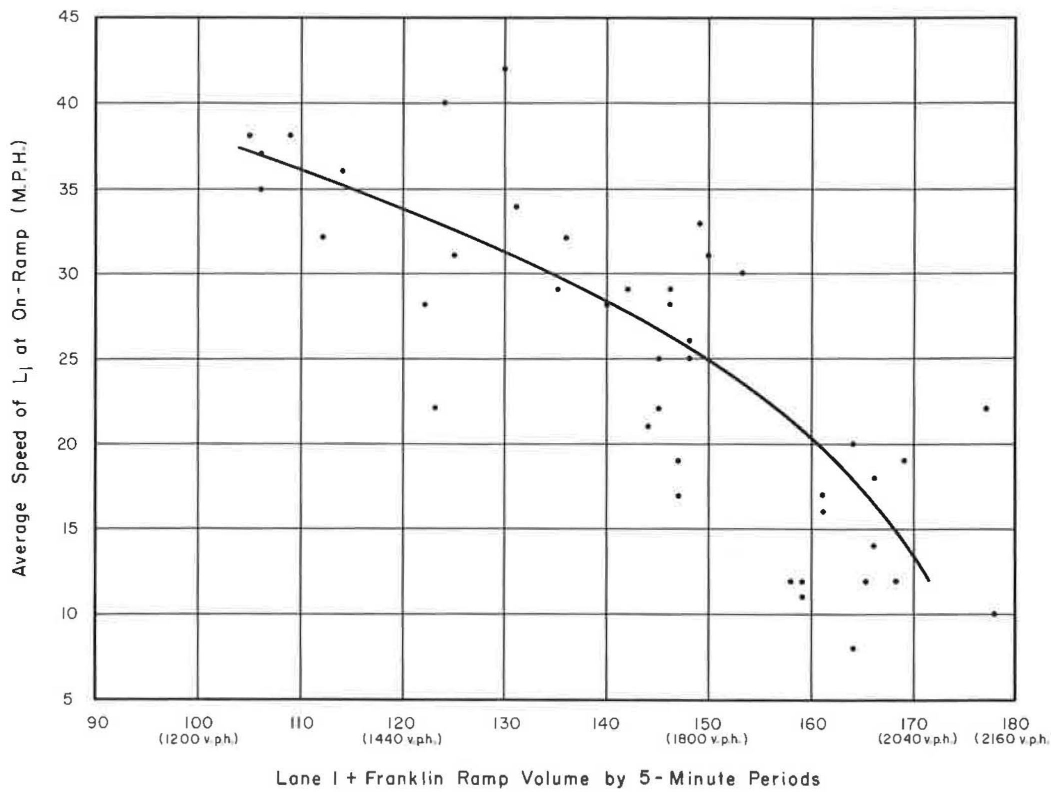


Figure 13. Speed of lane 1 vehicles related to total lane 1 and ramp vehicles.

the Eastshore Freeway, and several ramps on the Los Gatos-San Jose Freeway) combined ramp and lane 1 volumes of 1,800 vph scarcely reduce lane 1 speeds below normal rates. Average speeds would be 35 mph or more at these volume rates.

Because a more or less constant percentage of ramp traffic would shift out of the right lane regardless of the ramp volume, it might be assumed that the higher the ramp volume, the worse the effect will be on total freeway volume. Figure 14 shows that at a given total flow rate (including ramp traffic), the lower the ramp volume the better the operation. Between total volume rates of 5,400 and 5,800 vph there are twelve 5-min periods with ramp volume rates of 600 vph or less, and thirteen 5-min periods with ramp volume rates greater than 600 vph. Yet during 10 of the 13 periods with ramp rates greater than 600 vph, freeway speeds were 20 mph or less. During the periods of less than 600 vph ramp rates, all had freeway speeds of greater than 20 mph.

Figure 14 also indicates, although not conclusively, that the capacity of the section may be reduced when ramp volumes are high. The highest total volume rates did not occur when the ramp volume rates were highest.

The implications are fairly obvious. Essentially, two small volume ramps will result in better freeway operation than one large volume ramp, and possibly result in higher capacity. A better distribution among all freeway lanes will result and fewer vehicles will change lanes at one time or in one short section.

Therefore, the common (and expensive) practice of merging two ramps prior to a single merge to the freeway may not be the best design. As long as the distance between successive on-ramps is enough to permit an adequate acceleration lane, vehicles should merge into the freeway separately.

Vehicle Paths.—In studying traffic operation on the section, stoppages would be observed frequently in lane 1 at the on-ramp merge and occasionally in the other lanes.

Merging Traffic.—A momentary stoppage of lane 1 and/or on-ramp vehicles could occur any time the combined ramp-lane 1 volume rate for very short periods (about 30

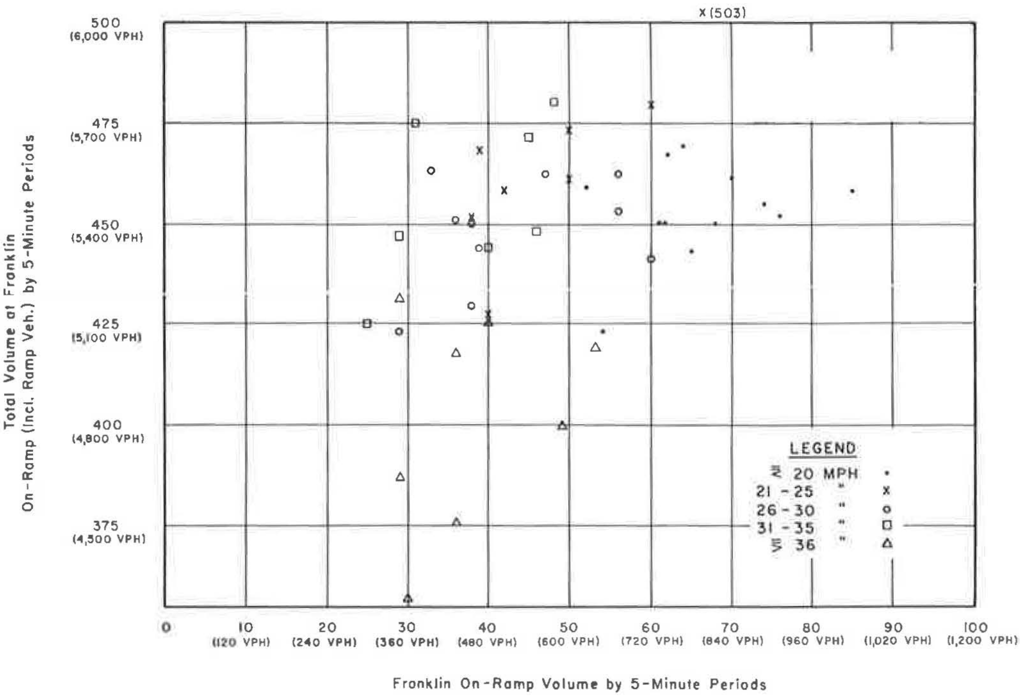


Figure 14. Average freeway speed at Franklin on-ramp, excluding ramp vehicles.

sec) was more than 1,800 vph. Because of statistical variations in headways, such stoppage occurs even during 5-min periods of relatively low volume flow. There were numerous ramp vehicles that stopped and waited for a gap at rates less than 1,800, not as a result of a capacity limitation or a lack of gaps to merge properly, but because of the effects of a short acceleration lane at low volumes.

If demand slackens, congestion dissipates quickly. If demand remains, congestion remains. If demand increases, congestion at the merge point stays the same, but the extra demand is reflected in longer back-ups. In each case the output rate (number able to merge) is the same. The actual number able to merge without congestion as well as the maximum that can merge depends on traffic characteristics and geometric conditions.

Figure 15 plots ramp and right lane vehicle paths for about a 2-min period containing a slight stoppage during a 5-min period at near capacity flow. (The 5-min period is prior to the peak and the full demand is able to reach this point; in other words, the freeway vehicles have not previously been stopped at any point upstream of the merge.) Although the paths in the middle portion of the section are approximate, every vehicle is accounted for.

At the Franklin on-ramp for the 130-sec period shown there are 67 vehicles on the ramp and lane 1, or a merging rate of about 1,850 vph, and 27 vehicles on the ramp (750 vph).

Of 27 ramp vehicles, 6 stayed in lane 1, 15 moved to the left, 2 got off at the Cahuenga Boulevard off-ramp, and 4 are unknown or unidentified. The unidentified ones had to either go to lane 2 or the ramp because all lane 1 vehicles at Cahuenga Boulevard were accounted for.

Of 40 lane 1 vehicles, 24 stayed in lane 1, 7 moved to the left including one who came back to lane 1 after passing a vehicle, 7 to the off-ramp, and 2 were unknown.

Of the 67 lane 1 and ramp vehicles at the ramp, 31 were still in lane 1 at Cahuenga Boulevard, 5 additional vehicles had entered lane 1 from lane 2. These probably were destined for the downstream Highland Avenue off-ramp.

For the 2-min period, lane 1 average speed at Cahuenga Boulevard was constant and about 35 mph. At the beginning and end of the 2-min period, merging was fairly smooth, and lane 1 vehicles traveled at an average speed of about 30 mph through the merge.

For the first 14 ramp and lane 1 vehicles, a high merging rate was maintained. The average headway was 1.42 sec or an equivalent volume rate of 2,500 vph, but it could not continue. One vehicle slowed to 14 mph behind an entering ramp vehicle. The entering ramp vehicle went to the Cahuenga off-ramp. If an auxiliary lane were available, the following vehicle would not have been forced to slow down, and much of the congestion for the following minute might have been eliminated.

Succeeding vehicles, including a large truck, also had to slow down. This congestion would have cleared up quickly except that a high arrival rate continued. From time 39 sec to 63 sec, 17 ramp and lane 1 vehicles arrived—an average headway of 1.5 sec.

The high flow rate could not be maintained and both lane 1 and ramp vehicles were forced to come to a stop momentarily. As demand slackened, good merging operation resumed within seconds.

The main point indicated by Figure 15 is that at high volumes these momentary stoppages are unavoidable and do not result in a loss of capacity (although a longer acceleration lane permits higher volumes before stoppages occur, and probably the lower the ramp volume the greater the total merging volume rate that can be maintained).

In such a plot showing individual headways, actions of a few vehicles are not statistically reliable. In other words, simply because vehicles 1 through 14 merge smoothly at a very high rate, it is not correct to say that the congestion during the minute from time 25 to 85 sec reduces capacity.

The input volume (ramp plus right lane vehicles) for the minute preceding time 25 sec, which includes the momentary high merging rate, is 27 vehicles (Fig. 15). The volume for the minute 25 to 85 sec, operating essentially under stop and go conditions, is 34 vehicles, including 4 trucks whose extra length is significant in this type of operation. During the minute following time 85 sec, speeds picked up to 35 mph, but volume dropped off to 23 vehicles because of momentary (statistical) drop in demand.

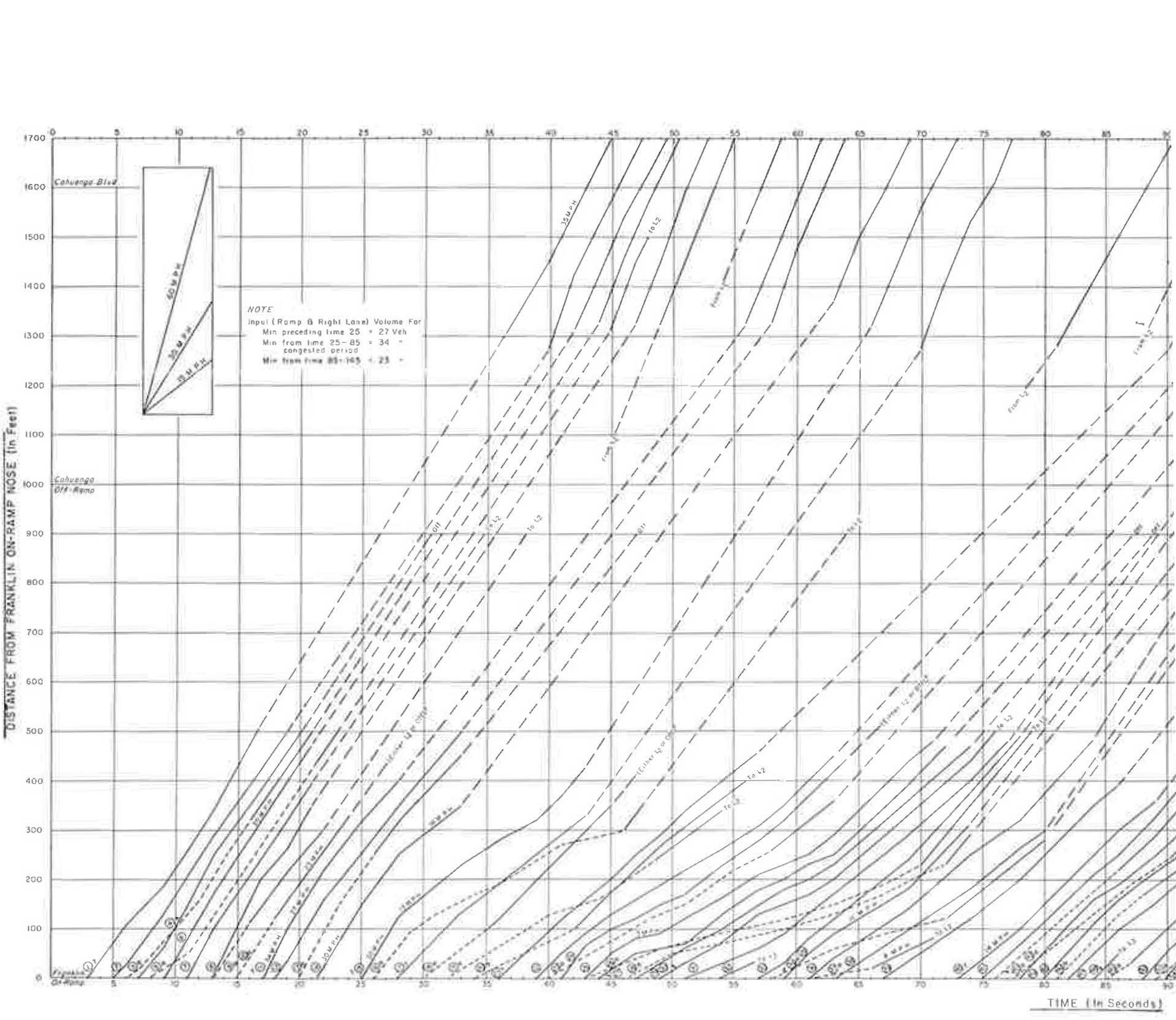
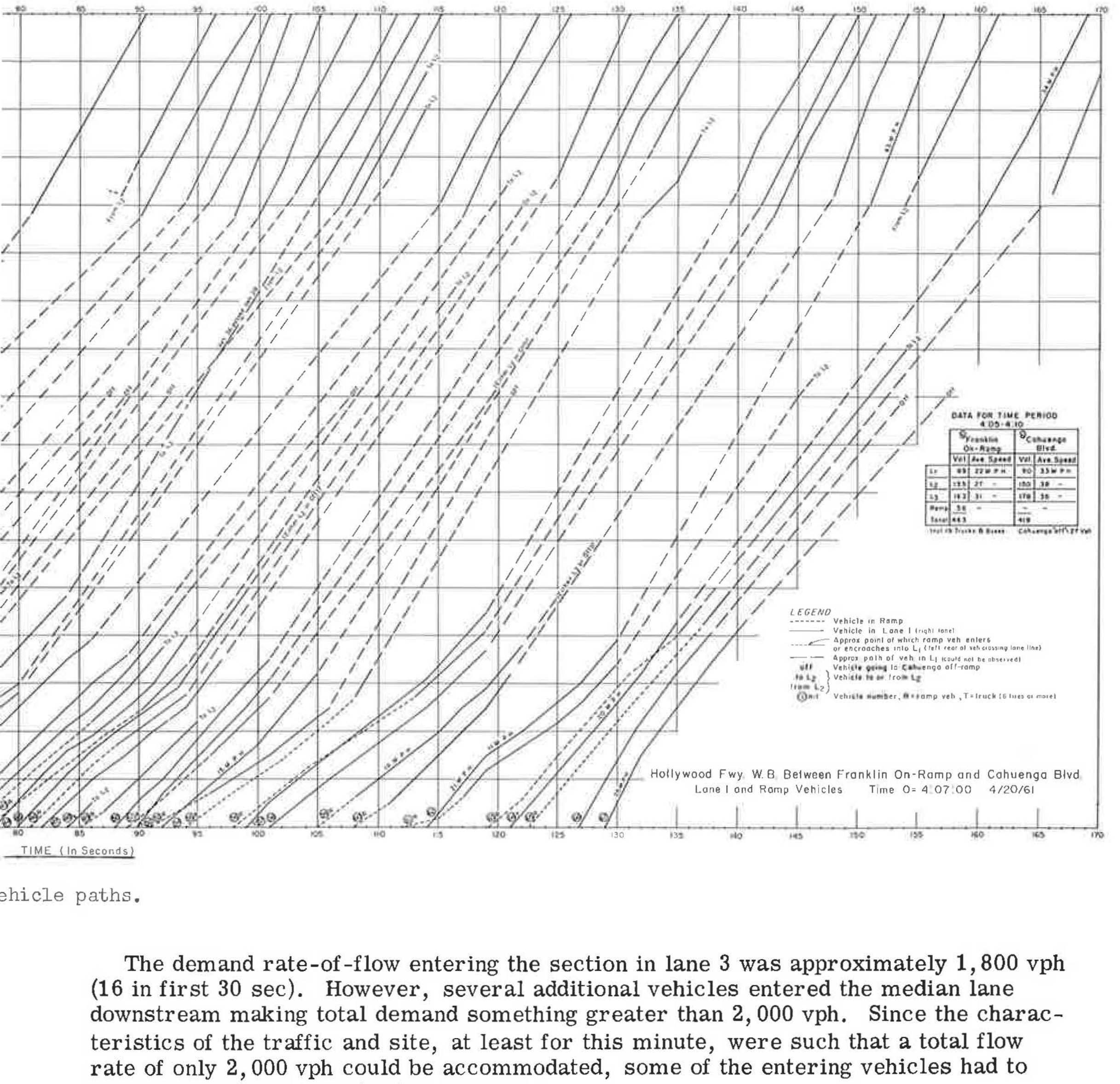


Figure 15

Extremely high merging rates can be accommodated at the merge, but only for short periods and/or over a very short distance. Drivers will not maintain the close spacings and will either slow down or shift to the left lanes—often whether there is room for them or not. Thus, as they shift, problems can be caused in the other lanes.

Median Lane. —The shifting traffic occasionally caused a stoppage in the median lane. In effect, the traffic moving into the median lane is the same as ramp traffic, and when merging rates exceed capacity, the same thing will happen as occurs at ramps. But because there is an infinitely long "acceleration" lane and "ramp" volume is small, stoppages do not occur until very high rates of flow are reached. (This is one of the reasons rates as high as 2,400 vph can be maintained in median lanes.)

Figure 16 plots vehicle paths in the median lane for about a 1-min period in which one of the stoppages or shock waves occurred. At the beginning of the period, flow was smooth although slow. As several lane 2 vehicles merged into the median lane, the following lane 3 vehicles slowed down until an actual shock wave was created. The first lane 2 vehicle merging into lane 3 enters a time headway gap between vehicles 4 and 5 of about 1.5 sec. Vehicle 5 cannot maintain this spacing with the entering vehicle in front, and therefore slows down so that he is 3 sec behind vehicle 4.



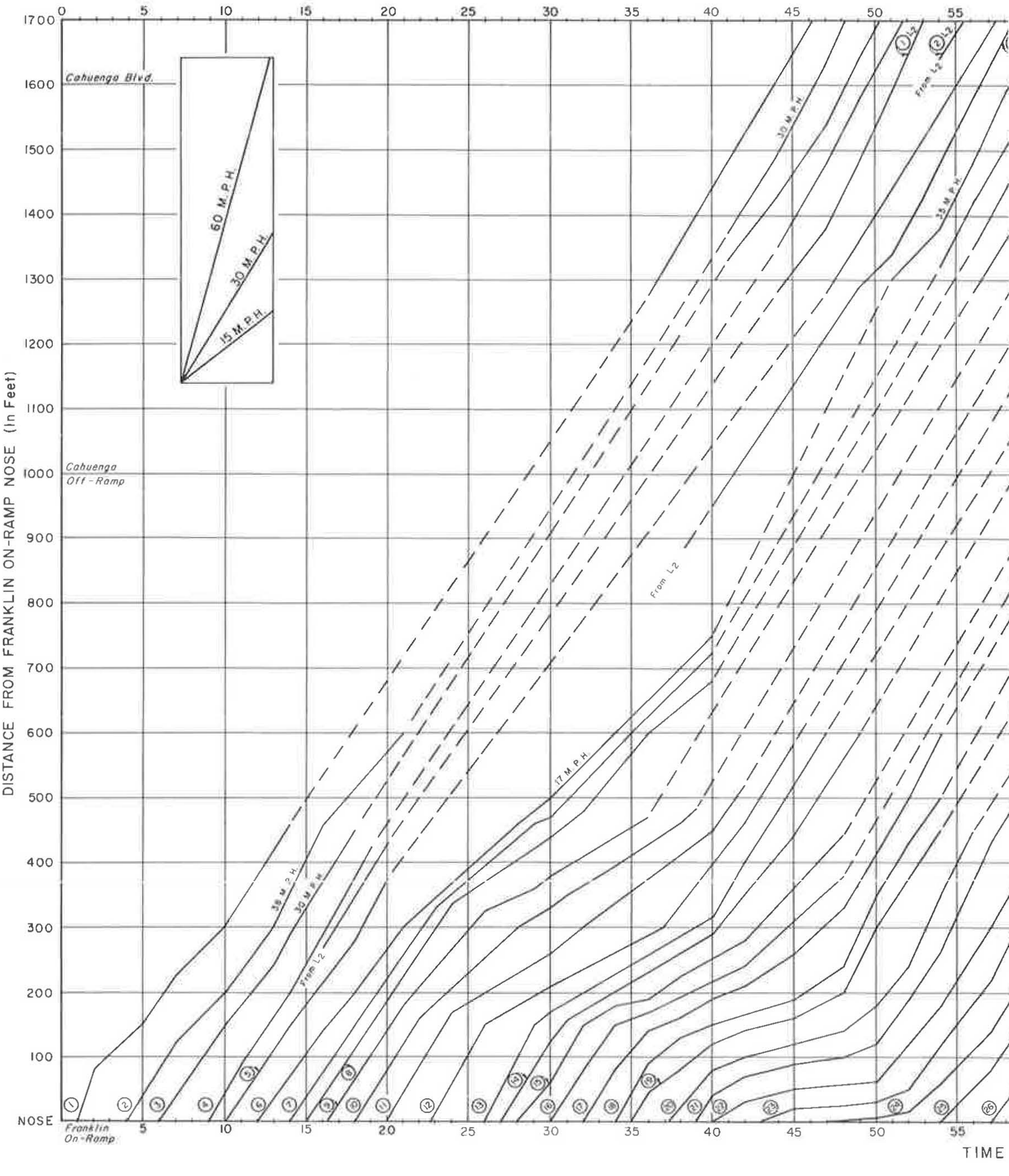
ehicle paths.

The demand rate-of-flow entering the section in lane 3 was approximately 1,800 vph (16 in first 30 sec). However, several additional vehicles entered the median lane downstream making total demand something greater than 2,000 vph. Since the characteristics of the traffic and site, at least for this minute, were such that a total flow rate of only 2,000 vph could be accommodated, some of the entering vehicles had to wait, thus creating the shock wave.

Therefore, it cannot be said that the minute volume rate was only about 1,650 vph because of the stoppage. This is not the full picture. Only because another 350 were entering downstream could 1,650 pass the entering point. At Cahuenga Boulevard the flow rate was still 2,000 vph.

Each vehicle in the shock wave was delayed from 6 to 10 sec. Once traffic moved again, normal flow resumed within seconds.

The shock wave essentially starts with vehicle 6 at time 20 sec about 300 ft downstream of the ramp nose. It reaches the nose at about time 40 sec (vehicle 22). Thus, the shock wave travels upstream at a speed of 15 ft per sec or 10 mph. The approach volume rate during the time the wave was traveling upstream was 1,900 vph.



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300
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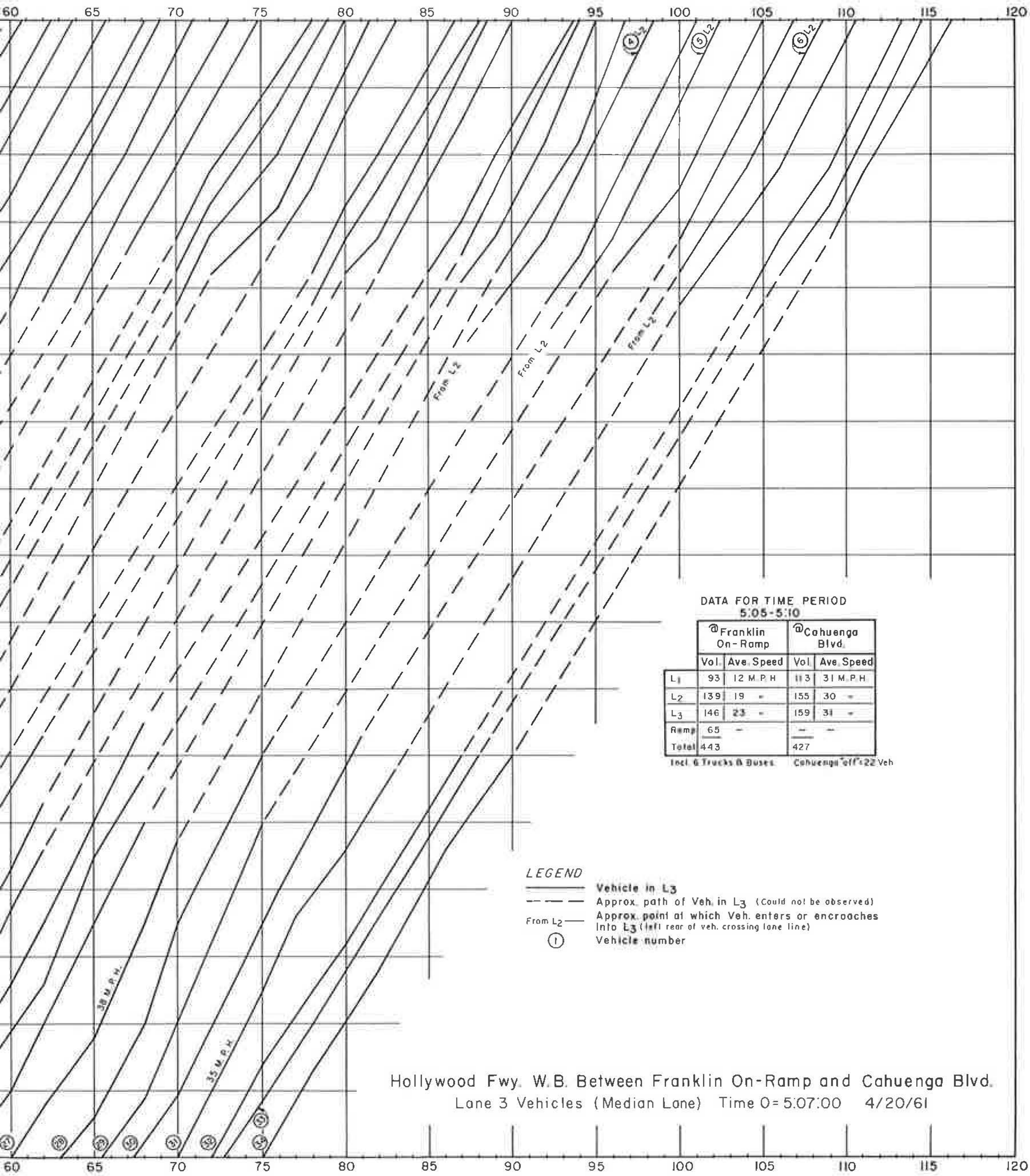
NOSE
Cahuenga Blvd.
Cahuenga
Off-Ramp
Franklin
On-Ramp
DISTANCE FROM FRANKLIN ON-RAMP NOSE (In Feet)
60 M.P.H.
30 M.P.H.
15 M.P.H.
30 M.P.H.
17 M.P.H.
From L2
TIME

Figure 16



DATA FOR TIME PERIOD 5:05-5:10

	@Franklin On-Ramp		@Cahuenga Blvd.	
	Vol.	Ave. Speed	Vol.	Ave. Speed
L_1	93	12 M.P.H.	113	31 M.P.H.
L_2	139	19 "	155	30 "
L_3	146	23 "	159	31 "
Ramp	65	—	—	—
Total	443		427	

Incl. 6 Trucks & Buses. Cahuenga "off" = 22 Veh.

n Seconds)

ehicle paths.

Notes on Freeway Capacity

KARL MOSKOWITZ and LEONARD NEWMAN, Assistant Traffic Engineers, California Division of Highways

•THE FIRST part of this paper deals with rural freeways, and strictly speaking is not a discussion of capacity. However, it is a discussion of operating characteristics at volumes less than capacity which will result in a "level of service," subjectively determined to be desirable for rural or long distance conditions.

Both design capacity and possible capacity of a uniform segment of an urban freeway are then discussed. A knowledge of these values is necessary to determine the basic number of lanes in the design of a freeway, and to review conditions on an existing freeway where traffic congestion occurs. Traffic flow cannot be increased by revising the design of one segment of freeway or interchange if the downstream freeway leg is operating at capacity, and delay cannot be reduced unless traffic flow is increased or diverted to another route.

A knowledge of capacity is necessary to recognize and pinpoint the bottlenecks. Because traffic often flows smoothly at a bottleneck, many observers make serious mistakes in identification and pinpointing. Conversely, even when a bottleneck is identified and a cure is proposed, it is necessary to know whether the upstream freeway can furnish enough flow to take advantage of the increased capacity and whether some new bottleneck will make its appearance at a downstream location.

The terminology, "Urban Freeways," does not mean that these capacities are not valid under rural conditions. Given the same geometry, driver, and vehicle characteristics, the capacity of a freeway is the same in a city, suburbs, or rural areas.

"Analysis of Interchanges" presents a procedure for reviewing the design or operation of a given geometric layout to be sure that it will work. Ramp capacity and weaving and merging capacities are defined and analyzed. The procedure may seem complicated at first, but weaving is a complicated problem. It is hoped that practicing designers will produce simplified tables, charts, and nomographs to aid in the solution of problems for specific cases, as well as for the general case. However, the complexity of the problem means that oversimplification must be avoided.

The discussion of "level of service" is necessarily subjective to some degree. Values in Table 1 were agreed to by the HRB Highway Capacity Committee in January 1962. These values will replace the "Rural Practical Capacity" values of the 1950 edition of the Highway Capacity Manual when the new edition is published.

Other values and all figures are based on extensive observations and intensive study of California Freeways during the past seven years (1955-62). Observations are continuing with the objective of refining the given values and filling in the blank spots.

The effects of grades, coupled with the proportion and speed distribution of slower vehicles, are not wholly understood, but Figure 2 represents the best available estimate of these effects. Research is under way which may cause some future changes in this figure. However, it is now based on enough facts and study to warrant the statement that it is far better than any individual opinion or summation of opinions. Effects of weather and lighting conditions are not treated at all, and this also represents a deficiency in present knowledge.

With these exceptions and others specifically pointed out this report may be considered authentic.

The Subcommittee on Definitions of the Committee on Highway Capacity adopted the following definitions at the January 1962 meeting of the Highway Research Board:

Paper sponsored by Committee on Highway Capacity.

The possible capacity is the maximum number of vehicles that can pass over a given section of roadway in one direction during one hour under specified traffic conditions.

Design capacity is the number of vehicles that can pass over a given section of roadway in one direction during one hour under specified traffic conditions and operating at a level of service. The level of service should be based on an engineering evaluation of the probability of traffic interruptions, on desired speed of operation as determined by trip purpose, type and location of the facility, the cost of vehicle operation, and by the cost of building, maintaining and operating the highway.

A design capacity is a volume generally selected for design purposes which will provide a desirable level of service.

RURAL FREEWAYS

On rural freeways, where most trips are long, the traffic volume during the design hour should be low enough to provide a reasonable degree of freedom of maneuver and absence of tension on the part of the drivers. This volume is quite low in comparison with the capacity of the freeway.

Even at extremely low volumes, there will be occasions where the projected time-distance graphs of three cars driving at steady speeds on a 2-lane one-way roadway will all reach a given point on the road at one time and a certain amount of adjustment of speed is required. The aggregate of such adjustments is negligible, in terms of psychological annoyance, up to values to be discussed. On grades, the aggregate or cumulative adjustments or conflicts are more frequent, but if the grades are short or if they are long distances apart, the cumulative tension for the trip is not increased very much. On the other hand, the capacity of any grade should never be exceeded.

On 4-lane freeways, with two lanes in each direction, it was found that at about 1,400 vehicles per hour (vph) in one direction on a level grade, the faster group of drivers began to be reluctant to use the right-hand lane for fear of being "trapped" behind a slow vehicle while an entire platoon of fast vehicles passes the slow vehicle. When rates exceed this number, this effect begins to be significant and the trapped vehicles will begin to break into the platoons passing in the left lane.

Curves showing speed versus traffic volume are not sensitive enough to pinpoint this effect. The fast platoons in the left lane are traveling 55 to 65 mph and the slow vehicles in the right lane are traveling 45 to 55 mph. The average speed of all vehicles is very slightly less than it is during low-volume flow. An observer standing at one location will note that long intervals go by between platoons, during which all cars are free moving, and then a platoon will go by in which the headways in the left lane are very short. It does not look like heavy flow, but about 50 percent of the drivers will be in a state of tension, driving bumper-to-bumper.

When there are three or more lanes in one direction, the probability of being trapped in the slow lane is reduced to negligible proportions at hourly volumes of less than 1,500 per added lane. It follows that for a given level of freedom, a freeway having three or more lanes in one direction will allow for a higher average hourly lane volume.

Table 1 may be used as a guide for determining the traffic volume which will result in practically unrestricted flow on various widths of freeway. Values are shown both for passenger cars only and for a normal percentage of trucks or slow vehicles. This percentage rarely exceeds 50 percent during the peak hour.

TABLE 1

PRACTICALLY UNRESTRICTED FLOW ON LEVEL GRADES, RURAL LONG-DISTANCE FREEWAYS[a]

No. of Lanes One Direction	Hr. Vol.—One Direction	
	No Trucks	5% Trucks
2	2,000	1,700
3	3,500	3,000
4	5,000	4,400

[a]Very high level of service.

The values given in Table 1 are not capacity volumes.[1] The only reason for listing them is to evaluate a quality of flow that will be acceptable for long-distance travel with almost complete absence of tension and to show the effect of additional lanes for this quality of flow. In deciding what value to use for design capacity, as previously defined, the length of highway involved, the distribution of individual trip lengths, and the cost of providing a given level of service should all be taken into account.

Effect of Grades

On sustained grades (more than ½ mi), the right lane will be pre-empted by trucks, and if it is desired to maintain a quality of flow on the grade equal to the quality on the level, it is necessary to add a climbing lane whenever the one-way volume exceeds 1,000 vph. However, because of economic factors, it may not always be desirable to do this.

There is a certain amount of platooning even on level roads at the volumes given in Table 1. When a plus grade is introduced, these platoons become more serious controls on capacity. The frequency of these platoons or bunches, the speed at which they move, and the possible capacity of the roadway itself are functions of (a) number of slow vehicles, (b) speed of slow vehicles (rate of grade), and (c) length of grade. If the grade is short and there are few trucks, there is a certain probability that there will be no trucks on the grade. If the grade is longer, there will be a greater probability that trucks on the grade will be encountered. Also, if the grade is steeper (and thus trucks slower), trucks will be on the grade a greater proportion of the time. Research linking these variables is now under way but is not complete.

For the time being, it may be assumed that grades of less than 2 percent and less than ½ mi can be disregarded, when considering flow rates less than possible capacity. Grades between 2 and 3 percent will form queues, but they will move fast enough so that high rates of flow can be maintained and the queues will not accumulate.

Pending the results of current research, the freeway capacity chart (see Fig. 2) may be used as a guide.

URBAN FREEWAYS

Fundamental Considerations

On a level urban freeway, when traffic flow is heavy enough to raise any questions regarding capacity, individual headways between vehicles vary from 0.5 sec up. In other words, in a very short interval of time and for a very few vehicles, the rate-of-flow in one lane or one file of vehicles can be as much as 7,200 vph. However, on the whole it is found that any 100 vehicles traveling through a significant distance, such as a quarter-mile or more, will not accept average headways of less than 1.8 sec, which is a rate-of-flow of 2,000 vph.[2] Some drivers in the total stream will accept lesser headways and these drivers tend to drive in the left-hand or median lane. For example, lane volumes in the median lane on many freeways consistently reach 2,200 vph. This does not mean, however, that all the vehicles in the stream (on all lanes) are willing to accept such short headways.

For design purposes this value (2,000 vph) should be reduced by 10 percent which results in the following rule: The basic fact about freeway traffic flow is that average

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[1] The Highway Capacity Committee used the following to describe Table 1: "(This table) may be used as a guide for determining the traffic volume which will result in a level of service where most of the cars will be affected by other vehicles in the stream, but the conflict is not unreasonable, even for long trips."

[2] Observations of extreme rates-of-flow exceeding this value have frequently been made, but rates higher than 2,000 vph cannot be considered dependable. The rate of 2,000 is capacity in the same sense that 4,000 psi is the compressive strength of a given concrete mix, even though the batch might produce individual cylinders varying from 3,500 to 4,500 psi at failure.

headways of less than 2 sec should not occur except during short intervals such as when a slug of traffic from a surface street traffic signal enters a freeway in about 30 seconds. During any 5-min interval, enough space should be provided so that no more than 150 vehicles will pass a point in one file. This may be referred to as a rate of 1,800 vph per lane for short periods.

Peak Hour Factor

In describing traffic flow, the motorist considers that failure occurs when traffic comes to a stop. This is a good enough definition for the traffic engineer and highway designer. A stipulated rate-of-flow for a 5-min period can insure that this will not occur, and with a 10-percent margin for error, this rate-of-flow is 1,800 vph per lane (average of all lanes).

However, the rate-of-flow for the highest 5-min interval of an hour is always higher than the rate-of-flow for the whole hour. This is because there is a natural statistical variability among the 12 five-min intervals, and there is also a variation in demand, owing to office and factory closing times, etc., within an hour, despite the metering effect of the surface street system.

The ratio of the rate-of-flow during the highest five minutes to the rate-of-flow during the whole hour is called the peak hour factor (PHF). For example, if there are 165 vehicles in the peak 5 minutes and 1,800 in the whole hour, the PHF factor would be 165 ÷ 150, or 1.1.

In large metropolitan areas, the peak 5-min rate-of-flow within an hour will be about 1.1 times the rate for a whole hour. For example, if the total hour volume were 1,800 vehicles per lane, the maximum 5-min rate-of-flow within the hour would be about 2,000 vph.

In smaller urban areas the peak 5-min rate-of-flow usually does not exceed 1.3 times the total hour rate.

It follows that if the volume in a large metropolitan area (PHF = 1.1) is predicted to be 1,800 per lane in a whole hour, and in a smaller area (PHF = 1.3) 1,500 per lane in a whole hour, the peak flow rates for short periods at both locations (and thus the probability of failure) will be about the same.

Urban Capacities

The preceding leads to the capacities given in Table 2 for a uniform segment of freeway, or "straight pipe" condition. These values are considered acceptable hourly operating volumes under "average" conditions. Average conditions are as follows:

1. Nearly level grade line (less than 2 percent).
2. About 3 percent trucks.
3. Absence of high-volume ramps in the vicinity which means straight pipe distribution of traffic among the lanes.

Acceptable volumes would be higher in the presence of one of the following factors:

1. Downhill grade line.
2. Less truck volume.
3. An "expanding" situation downstream. An expanding situation could be either the addition of a lane to the freeway, branch connection where the total number of lanes is increased and both legs have more than adequate capacity, or any other factor providing increased capacity.

Acceptable volumes would be lower in the presence of one of the following:

1. Sustained uphill grade line.
2. More truck volume.
3. Other factors causing mal-distribution of traffic.

Actually, average conditions may be considered hypothetical, and accepting operating conditions are not determined by the average, but rather by the sections of least capacity. For this reason, it is important for the engineer to exercise judgment and provide

TABLE 2

FREEWAY CAPACITY (HOUR VOLUME)[a]

Lane	Capacity (vph) 2 Lanes[b] 1.1 PHF	2 Lanes[b] 1.2 PHF	2 Lanes[b] 1.3 PHF	3 Lanes[b] 1.1 PHF	3 Lanes[b] 1.2 PHF	3 Lanes[b] 1.3 PHF	4 Lanes[b] 1.1 PHF	4 Lanes[b] 1.2 PHF	4 Lanes[b] 1.3 PHF	5 Lanes[b] 1.1 PHF	5 Lanes[b] 1.2 PHF
1 (rt.)	1,400	1,300	1,200	1,400	1,300	1,100	1,300	1,200	1,100	1,200	1,100
2	1,800	1,700	1,500	1,700	1,500	1,400	1,600	1,500	1,400	1,600	1,400
3	—	—	—	1,800	1,700	1,600	1,800	1,600	1,500	1,800	1,600
4	—	—	—	—	—	—	1,800	1,700	1,500	1,800	1,700
5	—	—	—	—	—	—	—	—	—	1,800	1,700
Total	3,200	3,000	2,700	4,900	4,500	4,100	6,500	6,000	5,500	8,200	7,500

[a] Queues will not develop and delay will be negligible.
[b] One direction.

a balanced design. The effectiveness of many miles of excellent design may be lost if adequate capacity is not provided for one or two short lengths.

Bottlenecks

Although Table 2 is useful in determining the basic number of lanes by freeway sections, it is not sufficient information to design an urban freeway. During the peak hours, operating conditions on urban freeways are a function of possible capacity of bottlenecks in the system which may or may not be dependent entirely on the number of lanes.

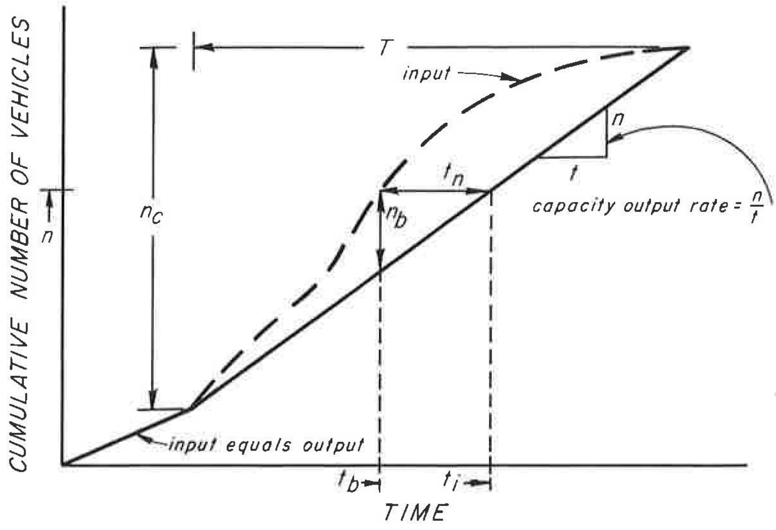
The traffic volume on an urban freeway will change at every entrance and exit ramp. Because of this, the ratio of demand to capacity varies from interchange to interchange. It is impossible to design a freeway so that this ratio will stay constant. Therefore, it is almost pointless to set up a lane-volume value in cars per hour to provide a given quality of flow along any significant length of highway. Driving along an urban freeway, even in a straight pipe condition between interchanges that are two or more miles apart, the individual driver encounters various instantaneous changes in conditions of flow. In one instant, he will be in the crest of a wave, and the next he might be in the trough.

When the input exceeds the capacity of a bottleneck, the freeway upstream from the bottleneck becomes a storage area and rate-of-flow in terms of cars per hour has no meaning. The rate-of-flow upstream of the bottleneck is independent of the geometric conditions at this location since it is bound to be equal to the rate-of-flow at the bottleneck.

Furthermore, when a bottleneck is operating at capacity, the speed of traffic upstream is also independent of geometric conditions on the upstream leg. The speed of traffic under such circumstances is a function of the excess of input over output and the length of time that the input rate has exceeded the output rate (Fig. 1).

When traffic is not backed up from a bottleneck, the average speed decreases somewhat as the rate-of-flow increases. The difference in speed is not significant in urban area capacity problems and should not be used as a criterion for determining acceptable operation. It should never be a consideration in establishing design speed. Design speed should be governed by operating conditions desired during off-peak hours. High standards of horizontal and vertical alignment will result in better operating conditions at very high volumes (even though speeds may be lower than design speed), and in greater safety at all hours of the day.

There are several conditions which can cause a bottleneck. The most frequent condition occurs where traffic is added to the mainline of the freeway without adding lanes to the mainline. This can occur at any entrance ramp along the freeway, and at a given total volume, is more likely to occur if the entering traffic is confined to a few high-volume ramps instead of several low-volume ramps. Another condition which can cause a bottleneck is a reduction in number of lanes. Other bottlenecks occur where the freeway begins an uphill grade.



n_b = No. of cars being delayed at time t_b, which is desired arrival time of nth car (nth car will arrive at upstream end of queue before this).

t_n = Delay suffered by nth car.

n_c = No. of cars delayed during entire period that queue exists.

Area between two curves = total delay in vehicle-minutes.

T = Total length of time that congestion lasts. Note that t_n, the delay suffered by any one car, is only a small fraction of T. Congestion may last 2 hours and the maximum delay to any one vehicle may be as little as 5 or 10 minutes.

If the average space headway in the queue is d, speed of nth car is a function of d, n_b, and t_n. Note that speed is dependent on capacity, not the other way around.

Figure 1. Relation between capacity and delay.

The problem is to define the locations of the bottlenecks and to provide adequate possible capacity at those locations. If this is done, the quality of service in between will take care of itself.

In a long straight pipe condition, traffic tends to distribute among the available lanes so that values such as given in Table 2 will apply. However, in the vicinity of bottlenecks, it is often found that distribution among the lanes does not follow the general pattern.

Bottleneck problems in general may be categorized as grade problems, where slow vehicles cause mal-distribution of traffic among the lanes, and merging and weaving problems at interchanges.

Grades

Figure 2 shows various levels of service as affected by long grades and a normal percentage of trucks. Although the precise effect of grades is not known, this may be used as a guide in evaluating grade problems for the time being or until further research requires a change.

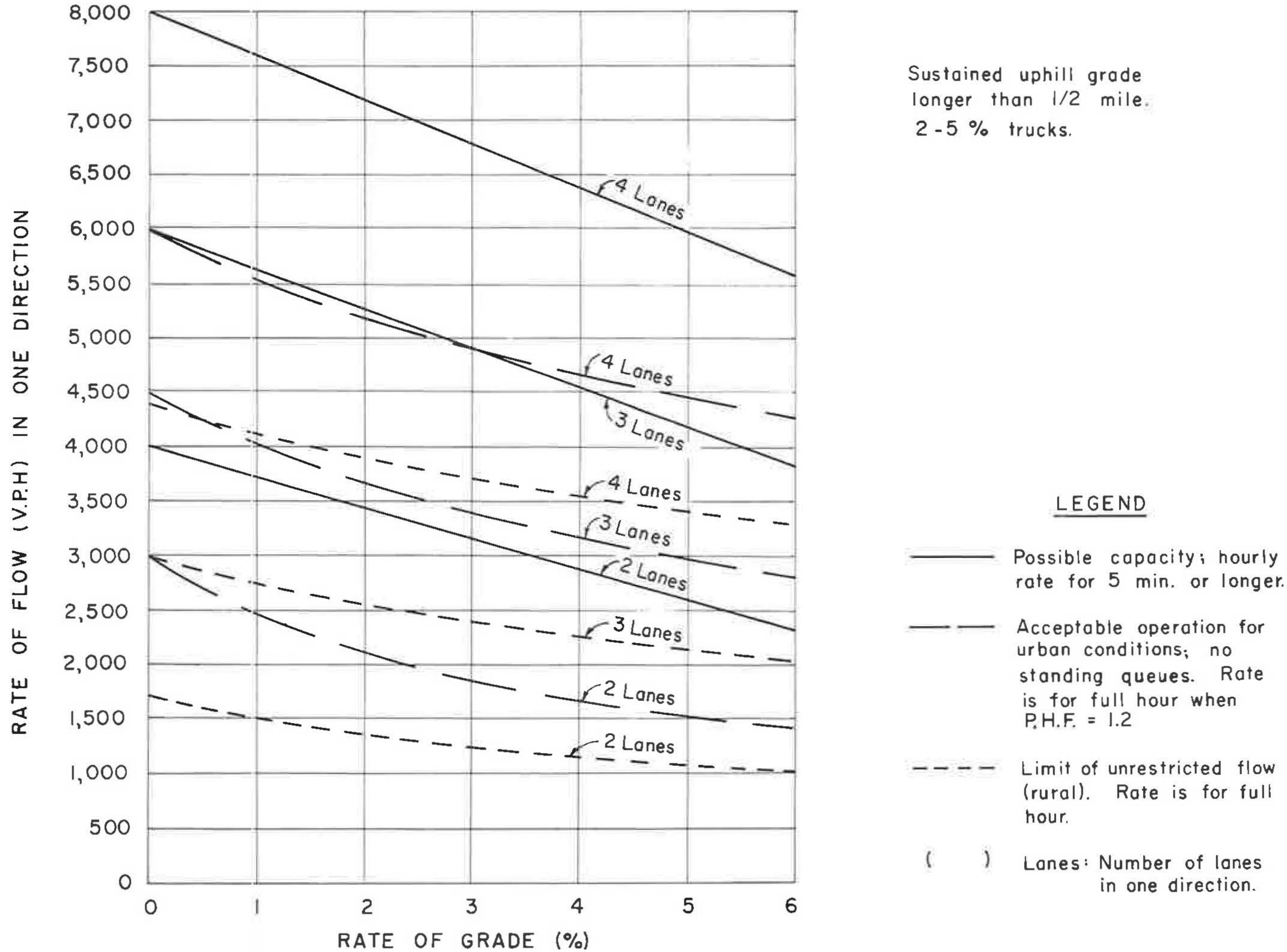


Figure 2. Freeway capacity.

ANALYSIS OF INTERCHANGE CAPACITY

The analysis of interchange capacity is essentially the analysis of conditions at ramp terminals.

Ramp Capacity

The rate-of-flow that an on- or off-ramp proper (turning roadway) can handle is about the same as a freeway lane or about 1,800 vph. Whether the ramp volume can be accommodated at the intersection with the surface street is a separate problem and should be analyzed as a regular street intersection problem.

When capacity is a consideration, any on-ramp roadway more than 1,000 ft long should be 2 lanes wide even when it is funneled to 1 lane at the merge. This allows passing and breaking up of queues and large gaps, thus permitting a more even arrival rate at the freeway and at higher speeds.

On an off-ramp, the amount of 2-lane roadway (or wider) beyond the exit nose is dependent primarily on capacity requirements at the surface street connection and storage space required.

The freeway terminals of ramps should be of standard design. The standard entrance ramp must provide (a) adequate merging distance for high speeds as well as low speeds at every location, (b) in combination with the approach ramp, adequate length for entering cars to accelerate from any turning speed, and (c) adequate merging distance for low volumes as well as high volumes.

Freeway to freeway connections are essentially the same as ramps and can be analyzed in the same manner. The turning roadway may be of a higher standard to permit higher speeds, but the terminals would be the same. The connections would be different only if the exit or entrance volumes were so high as to require dropping or adding a lane to facilitate 2-lane exits or entrances.

Two-lane ramp connections to the freeway are not generally used unless a lane is added or dropped, but in some cases, they are desirable even when a lane is not added or dropped. This could be the case when the ramp and freeway peak occur at different times. If 2-lane entrance ramp terminals are used, a parallel lane should also be provided for a substantial distance, in addition to the standard ramp taper, so that a portion of the ramp traffic will have a chance to move to the left before the remainder has to merge. Conversely, 2-lane exit ramps require a parallel deceleration lane in order to provide sufficient volume to utilize the lanes.

Calculating Weaving and Merging Capacities

As a first step in the design of a length of freeway, the number of lanes required is determined from the predicted hourly volume for the design year. For example, if the one-way hourly volume is predicted to be 6,000 vehicles, 4 lanes would be provided since an average of 1,500 vehicles per lane is within the limits of acceptable operations for 4 lanes (Table 2).

As a second step, flow by lanes must be checked in the vicinity of ramps. The following stipulations must be met (assuming grades of less than 3 percent and about 3 percent trucks):

1. Rate-of-flow in the right lane or auxiliary lane of a freeway or in a single-lane ramp should not exceed 1,800 vph.
2. Number of weaving vehicles should not exceed 2,100 vph in any 500-ft segment of a weaving section.
3. Average rate-of-flow across all lanes should not exceed 1,800 vph per lane.

As long as demand rate-of-flow (for 5 to 15 min) does not exceed the given limits, queuing or shock waves will not occur and operation upstream of the critical section will take the characteristics of straight pipe flow.

The described procedure only determines whether a certain volume level and traffic pattern will give acceptable operation; it does not evaluate quantitatively how much better operation would be for a certain lower volume level. The method is intended to be used to check a critical section to insure that it will work and not become a bottleneck for the predicted volume levels and traffic patterns or at least so that the limitations of the section will be realized.

Under normal conditions of straight pipe flow where there are no high-volume ramps in the vicinity, the lane distribution at near capacity conditions could be expected to be approximately as given in Table 2. Capacities might be reduced because traffic desires might be such that the general straight pipe distribution will not occur and an inordinate number of vehicles will try to use a single lane. Problems such as this occur, for example, at heavy volume ramps where a substantial portion of the traffic wants to be in the right lane and there is not enough traffic that will use the efficient high-capacity left lanes. (However, solving this problem by using left-hand ramps should not be attempted.)

Therefore, after the basic number of lanes and geometric design have been determined through the use of total-volume flow rates, lane distributions should be checked at any point where a bottleneck condition might be suspected.

Because rates-of-flow within an hour are higher than the flow for the full hour, the short-time rates of flow should be used in checking a section of freeway for its adequacy. Converting the full-hour volume to short-time flow rates is done by applying the PHF. All of the volumes or flow rates in the following refer to short-time rates.

Merging operation will be smooth as long as total ramp and adjacent lane rate-of-flow does not exceed 1,800 vph, provided that the entrance ramp terminal is long enough and has a gradual taper.

Maximum combined flow-rates for a merge of a particular ramp and adjacent freeway lane have been observed as high as 2,000 and 2,200 vph. However, it is not recommended that this value be anticipated in design procedures, since there are certain conditions of geometric design and traffic characteristics (which are difficult to predict or evaluate) that can prevent its attainment. A dependable figure is 1,800 vph which can be counted on under almost all circumstances, with normal truck percentages and grades of less than 3 percent.

Merging operation will vary considerably depending on the relative proportion of traffic on the ramp and adjacent lane. The smaller the number of ramp vehicles compared to adjacent lane vehicles (with the sum of the two being 1,800 vph), the better the merging operation. Entering ramp vehicles tend to move at slower speeds than freeway vehicles and often tend to arrive in platoons because of signal control. Thus, they are not as well spaced as freeway traffic, which causes higher instantaneous merging flow than would occur if ramp traffic arrived randomly. This also means that in most instances, two ramps of 400 vph each, will operate better than one ramp with a rate-of-flow of 800 vph.

In any case, regardless of the relative volumes, a combined flow rate of 1,800 vph will result in satisfactory operation. Operating conditions when this criterion is met will be such that average speeds (over the entire length of the merging area) will be between 30 and 40 mph.

Many times on a heavy-volume ramp the rate-of-flow on the ramp itself for 30 sec or a minute will be 1,800 vph, even though the flow rate over 5 or 10 min is only 800-1,000 vph. When this platoon arrives at the freeway, and if there are any vehicles in the adjacent freeway lane (as there almost always will be), severe reductions in speed will occur. If two cars arrive at the same spot at the same time, one will have to adjust its speed. It is a statistical certainty that will will happen at a ramp at almost any volume level—not as frequently at the lower volumes, but it will occur. This type of operation at ramps must be expected and not considered a failure in freeway operation. It cannot be designed out by assuming lower design capacities. Failure occurs when the queue does not dissipate, i.e., when the queue is continuous for several minutes.

This value, 1,800 vph (or an average headway of 2 sec) in any 5-min interval, is also the key for testing weaving lanes. In addition, the weaving that will take place in a short length must be checked. No more than 2,100 vph weaving should be permitted in any 500-ft segment of roadway, regardless of the number of lanes provided. (Weaving vehicles are defined as those that must actually cross paths; at least two lanes must be available and all weaving vehicles must cross the line—"crown line"—separating the two lanes.)

Possible capacity of a 500-ft length is about 2,300 vph but as in the case of possible capacity for merging traffic, it should not be counted on. Under most circumstances, 2,100 vph weaving in 500 ft can be reasonably expected. Speed and acceptable weaving volume are not directly related. Assuming a lower speed will not make the acceptable weaving volume higher. A given weaving volume will operate much more smoothly at high speeds than at low speeds.

Ordinarily, if the 1,800 vph in any one lane requirement is met, weaving volume will not be a control when the length available for weaving is 1,500 ft or more.

Examples of Procedure

The following examples illustrate the procedure and basic facts which are used to determine the lane distribution on a critical portion of the freeway so that the described procedure can be accomplished.

An 8-lane freeway with an on- and off-ramp is assumed, as shown in Figure 3. One-way traffic upstream of the on-ramp is at a rate of 5,500 vph. It will be developed that with 5,500 vph on the main line approaching the on-ramp merge, including 700 going to the off-ramp, 1,200 of the 5,500 will be in the right lane at the nose of the on-ramp. Since an auxiliary lane is not provided, all of the on-ramp vehicles must merge with this 1,200. Since rate-of-flow in a merging lane should not exceed 1,800 vph, 600 vph is the maximum rate-of-flow that may enter from the on-ramp.

If the off-ramp were a greater distance away from the on-ramp, then not all of the 700 off-ramp vehicles would be in the right lane, thus leaving room for more on-ramp vehicles. The improved distribution of traffic across all lanes would result in a higher capacity on the freeway between the on- and off-ramp.

If the ramps were 2,000 ft apart, then about 550 of the 700 off-ramp vehicles would be in the shoulder lane, thus leaving room for an additional 150 vehicles from the on-ramp (Fig. 4).

It is now assumed, in the case where the ramps are 1,000 ft apart, that the on-ramp has a demand of 1,200 vph. As illustrated, only 600 can be absorbed efficiently be-

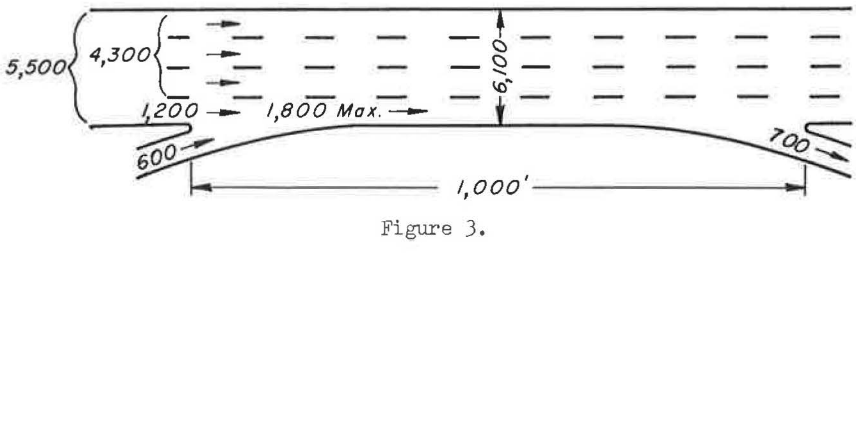


Figure 3.

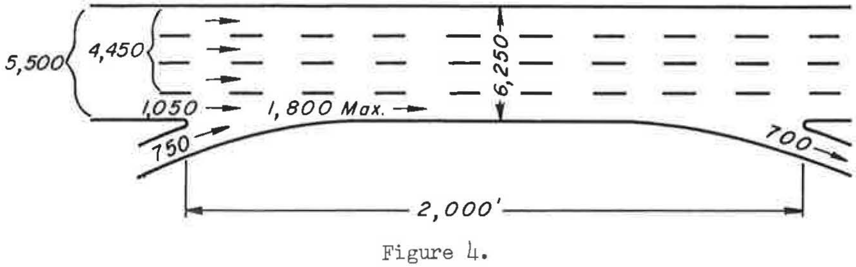


Figure 4.

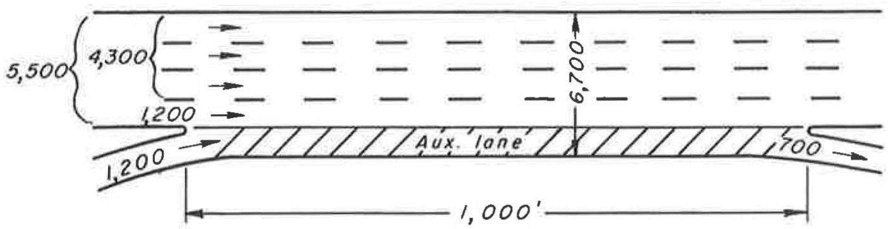


Figure 5.

cause there are 1,200 in the right lane already.[3] But if an auxiliary lane is provided between the two ramps, then the off-ramp vehicles can move to the right before on-ramp vehicles have to merge into the main stream. The on-ramp can absorb 1,200 vph, because lane changing is such that there will be no more than 1,800 vph at any point in the auxiliary or right lane. Therefore, by adding the auxiliary lane the capacity of the ramp is greatly increased (Fig. 5).

As previously stated, the principle is that traffic volume in a merging or weaving lane at any point should not exceed 1,800 vph.

The basic problem in implementing this procedure is to know how traffic will distribute across the freeway lanes.

Distribution of Traffic by Lanes

Traffic at a point on a freeway can be divided into three segments:

1. Through traffic—traffic not involved in ramp movements within a distance of 4,000 ft.
2. On-ramp traffic—traffic which has entered the freeway a certain distance upstream of the point or section under study. This distance is a variable to be put into the problem.
3. Off-ramp traffic—traffic destined for an off-ramp a certain distance downstream of the point or section under study. This distance is also an input variable.

Under most conditions, when capacity volumes are approached, each of these segments, which make up the total freeway flow, will be distributed in accordance with the curves in Figures 6, 7, and 8 (or Fig. 9 in lieu of 7 and 8).

The distributions presume the existence of demand for near-capacity volumes in the right lane at the point being considered. Unless there are about 1,800 vph total, in the right lane, the distribution is not necessarily valid. For example, assuming through traffic at a certain point on a 4-lane section (one-way) is 6,000 vph, Figure 6 would place 10 percent or 600 in the right lane. This is true provided that ramp vehicles will bring the total volume in the right lane at this point close to 1,800 vph. If ramps are so far removed from this point that little ramp traffic would be assigned to lane 1, then the 10 percent of the through traffic assigned to lane 1 would be too low. However, if the volume in the right lane comes out to be considerably less than 1,800 vph, then the section is obviously satisfactory and the actual distribution is of no significance. That is to say, the figures are valid when checking capacity conditions. For situations where volume is well below capacity, they are irrelevant.

The figures were developed from examination of actual cases operating satisfactorily. Additional research is being conducted to further verify and refine them, and to extend their range of application. Several examples comparing calculated volumes in the right lane with actual observed volumes are given in Appendix A.

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[3] The fact that only 600 can be absorbed efficiently does not mean that only 600 will get on the freeway. With a demand of 1,200, the difference of 600 will be partly waiting in a queue on the ramp, and partly in a queue on the freeway. The freeway flow will have broken down with long irregular queuing, mostly in the right lane but with spill-over queuing and stop-and-go operation in adjacent lanes. This type of operation results in hazardous lane-changing upstream.

The figures are intended for use with single-lane on- and off-ramps with or without an auxiliary lane between them. They will also be used for the more complex situations involving 2-lane ramps and branch connections. However, they may require some modifications and are currently under study. This procedure should not be used for left-hand ramps.

Limited observation indicates that the combined rate-of-flow for the left lane and a left-hand on-ramp of 1,800 vph will provide acceptable operation as in the standard right-side ramp. However, when the average volume on all lanes is 1,800 vph, smooth flow on the freeway between interchanges requires that the left lane be carrying high-volume rates of 2,000 vph or more. Left-hand ramps would cut this to 1,800. The difference could not be made up in the other lanes as volume rates in the right-hand lane would still be limited to 1,800 vph to maintain good operation. This capacity re-

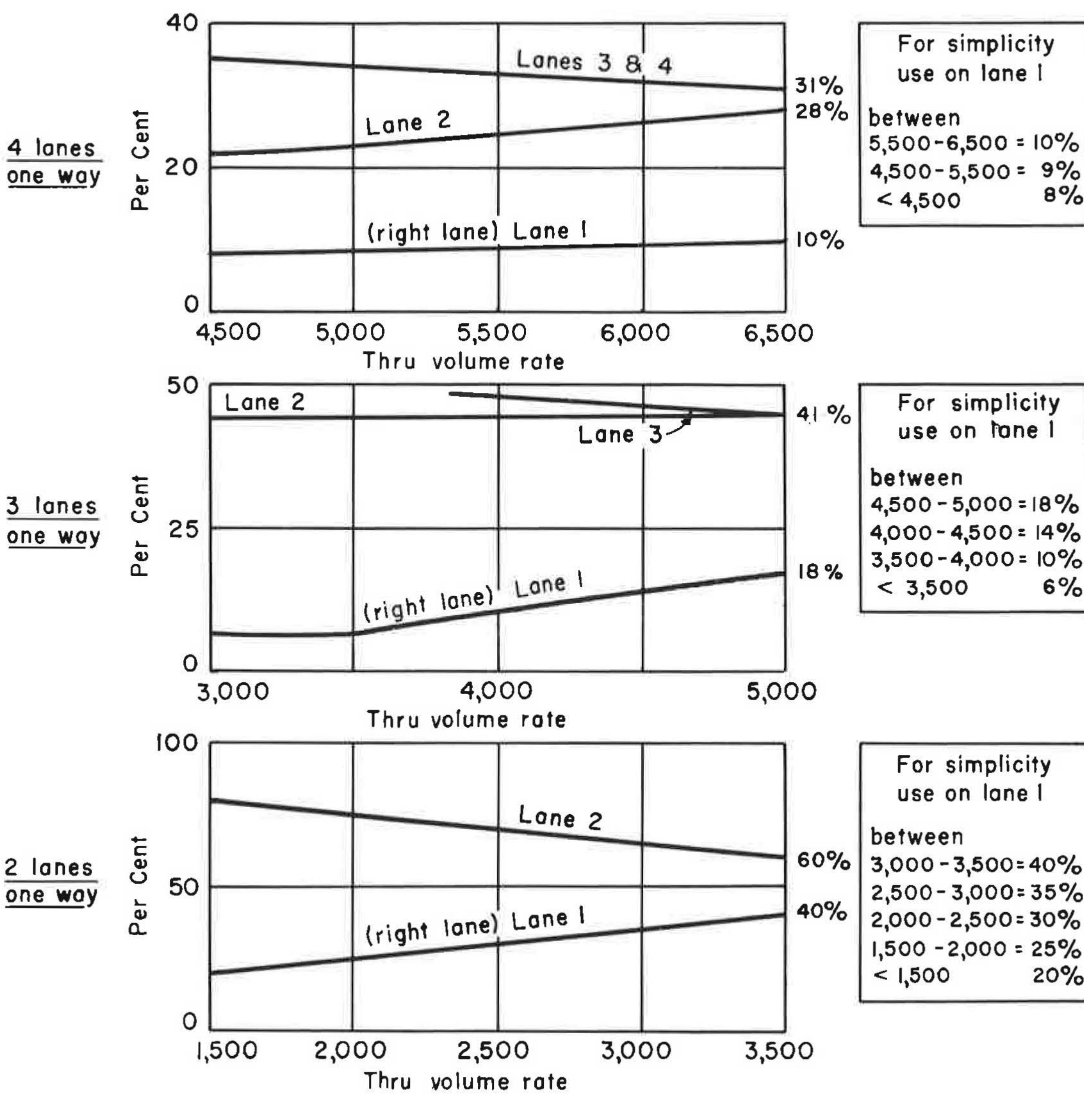


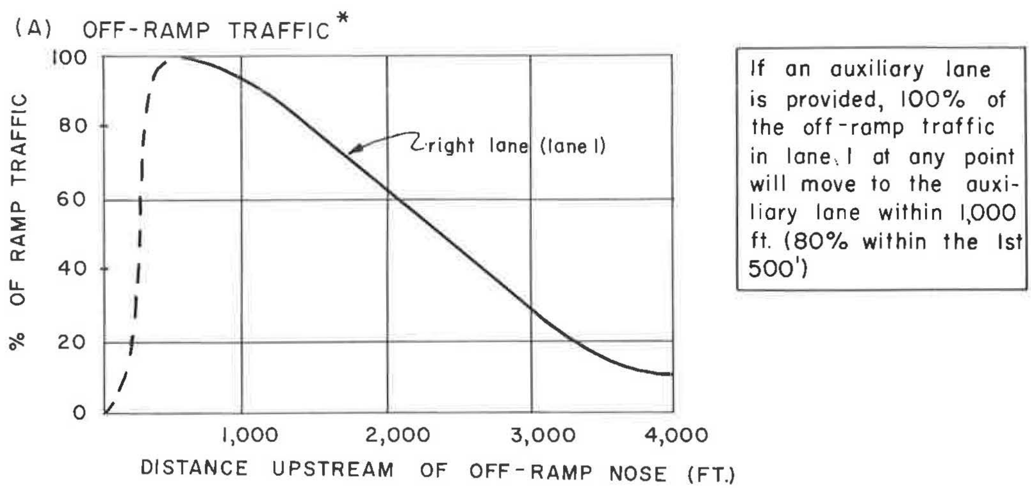
Figure 6. Distribution by lane of one-way through traffic, not involved in a ramp movement within 4,000 ft (percentages are not necessarily distributions under free flow or light ramp traffic, but under pressure of high volumes in right lanes).

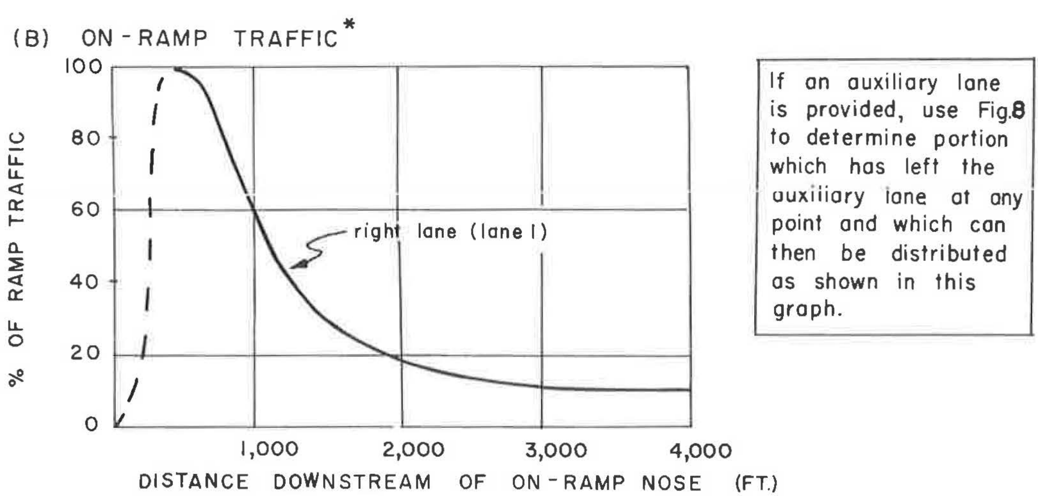
duction is in addition to other undesirable operational characteristics of left-hand ramps.

Figure 6 indicates the number of through vehicles that will stay in the right lane even though they are not involved in a ramp movement and are likely to be forced to adjust their speeds because of ramp maneuvering and statistical distribution of ramp traffic headways.

For example, assume 4 lanes one-way and 6,300 vph through traffic (which is defined as traffic not involved in a ramp movement within 4,000 ft). Reading from the graph, 10 percent, or 630 vph, will be in the right lane.

Figure 7 (A) shows the percentage of the off-ramp traffic in the right lane at any distance upstream of the ramp. The curve indicates that in the case of a conventional off-





* The percentage of ramp traffic in the right lane cannot be less than the percentage of thru traffic in the right lane (from Fig. 6). If the % comes out less, ramp traffic should be considered as thru traffic.

Figure 7. Percentage of ramp traffic in right lane (percentages are not necessarily distributions under free flow or light ramp traffic, but under pressure of high volumes in right lanes).

ramp (no auxiliary lane—a standard taper), 100 percent of the off-ramp traffic will be in the right lane at a point 500 ft upstream of the off-ramp nose. At a point 2,000 ft upstream of the nose, 63 percent of the off-ramp traffic will be in the right lane.

Figure 7 illustrates an important point in connection with an ordinary off-ramp. Because there is always some through traffic in the right lane, it would not be possible to supply 1,800 vph to an off-ramp even though the ramp might handle it. But if a parallel lane were added (an auxiliary lane in effect), 1,800 could be supplied to a ramp. For example, assume the following conditions: off-ramp demand is 1,800 vph, 350 vph going through in the right lane, and a parallel lane 1,500 ft long. At the beginning of the parallel lane (1,500 ft upstream of the off-ramp nose), 79 percent of the ramp traffic or 1,420 (0.79 × 1,800 vph) would be in the right lane. This combined with the 350 vph thru volume, a total of less than 1,800 is satisfactory. Then off-ramp traffic as it

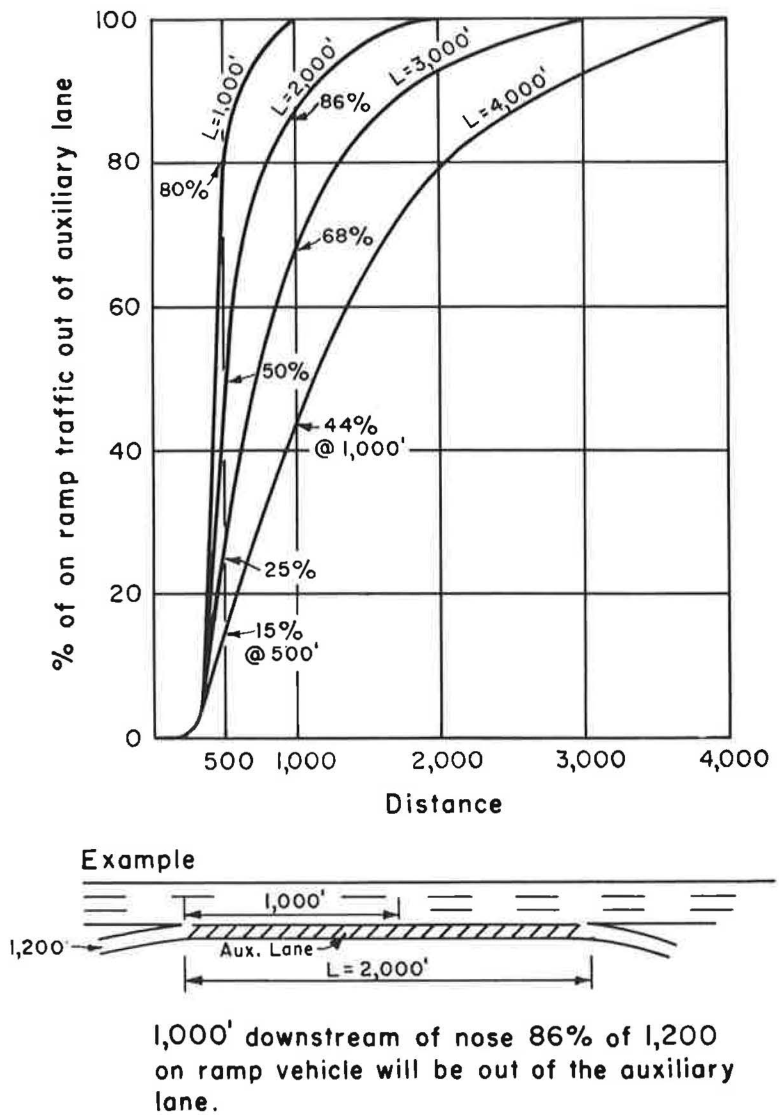


Figure 8. Percent of on-ramp traffic leaving auxiliary lane at any point for a given length of auxiliary lane (L).

progresses downstream will move into the parallel lane leaving room for the remaining 21 percent of the off-ramp traffic to move to the right lane. This effect has been observed at heavy off-ramps where cars create a parallel lane by riding the shoulder previous to the off-ramp deceleration lane.

Figure 7 (B) shows the percentage of on-ramp traffic in the right lane at any point downstream of the ramp. For example, 500 ft downstream of the on-ramp nose, 100 percent of the ramp traffic will have encroached on the right-hand freeway lane. The whole vehicle may not be in lane 1, but the left side will be close enough to create a headway unit in lane 1. One thousand feet downstream of the nose, 60 percent will be in the right lane with the other 40 percent having moved over to the left if there is room in the other lanes.

If auxiliary lanes between ramps are provided, basically the same system is used. In the case of off-ramp traffic, all off-ramp traffic in lane 1 at any point will move into the auxiliary lane within 1,000 ft (with 80 percent moving over within the first 500 ft). For example, assume an on- and off-ramp 1,000 ft apart with an auxiliary lane. As shown in Figure 7 (A) abscissa 1,000 ft, 95 percent of the off-ramp traffic will be in the right lane at the on-ramp nose. Five hundred feet downstream, 80 of the 95 percent will have moved over to the auxiliary lane leaving 19 plus the remaining 5 percent of the off-ramp traffic (100 minus 95 percent) in the right lane (see Fig. 9).

In the case of on-ramp traffic where an auxiliary lane is provided, Figure 8 should be used in conjunction with Figure 7. Figure 8 shows the manner in which ramp traffic leaves the auxiliary lane. For example, assume adjacent on- and off-ramps 1,000 ft apart. Figure 8 indicates that 500 ft downstream of the on-ramp nose, 80 percent of the ramp traffic will have moved to L_1. The traffic which has moved to the right lane is then distributed using Figure 7, which indicates that 60 of the 80 percent will still be in L_1 1,000 ft downstream of the on-ramp nose (see Fig. 9).

With these three figures, various traffic demands and geometric conditions involving adjacent ramps with or without auxiliary lanes can be checked to determine whether they will operate at acceptable levels, i.e., no more than 1,800 vph in the right lane or auxiliary lane.

Weaving volumes that take place in any 500-ft segment can also be determined from these graphs.

Figure 9 shows the distribution of ramp traffic at 500-ft spacings for several general cases. It is calculated from Figures 7 and 8 and makes it easier to solve general problems. For example, assume on- and off-ramps 1,000 ft apart with an auxiliary lane and the following traffic pattern: L_1 thru = 300 vph; on-ramp = 1,000 vph; off-ramp = 1,200 vph (and no on-ramp to off-ramp traffic). The critical point is at the 500-ft section. At this point, traffic in L_1 will be 300 (L_1 thru) plus 80 percent of the on-ramp traffic or 800, and 24 percent of the off-ramp traffic or about 300—a total of 1,400 which is satisfactory.

The weaving that takes place in a 500-ft section can also be determined. In the same example, in the first 500 ft, 80 percent of the on-ramp traffic will weave with 76 percent of the off-ramp traffic. This would be $(0.80)(1,000) + (0.76)(1,200) =$ about 1,700 vph which is satisfactory.

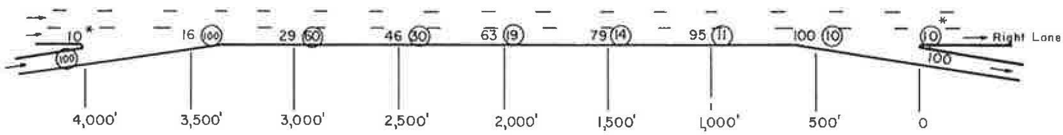
Obviously, in actual practice there are few weaving sections with lengths that are exact multiples of 500 ft. However, the length of the section under investigation can be rounded to the nearest 500 ft, without exceeding allowable error in estimating the acceptability of traffic operation.

GENERAL REMARKS

To obtain maximum flow and good operation on the freeway, traffic needs a minimum of 600 ft to change lanes. Therefore, in addition to controls imposed by lane distribution of traffic, if vehicles must merge and then move to a second through lane (as in the case of a 2-lane off-ramp), the minimum distance between "paint" noses should be 1,200 ft regardless of the lowness of the weaving volumes. Since the paint nose, or actual confluence point, is offset several feet laterally from the concrete nose, the distance (on a flat taper) between the paint nose and the concrete nose is several hundred ft. The distance between concrete noses is seldom less than 1,800 ft (Fig. 10).

(25) = % of on-ramp to fwy traffic
15 = % of fwy to off-ramp traffic

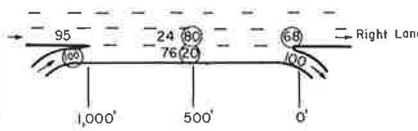
CASE I Single lane on-and off-ramps w/o auxiliary lane



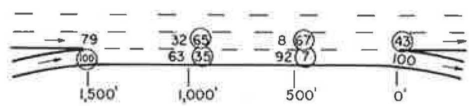
CASE II Single lane on-and off-ramps with auxiliary lane

(a) L (length of aux. lane btw. concrete noses) = 1,000'

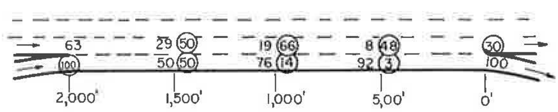
Example
Given: L = 1,000', L_1 thru (from Fig 6) = 300 vph,
on-ramp = 1,000 vph, off-ramp = 1,200 vph
on-ramp to off-ramp = 0
Find: Vol. in L_1 @ 500' = 300 + (.80)(1,000) + (.24)(1,200) = 1,390



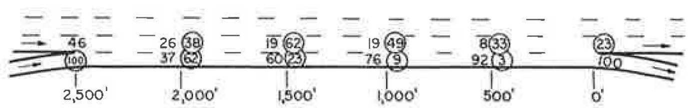
(b) L = 1,500'



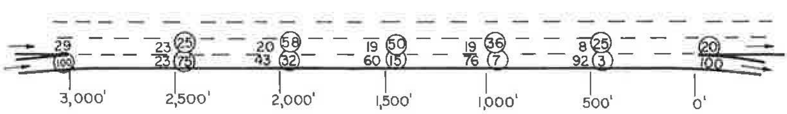
(c) L = 2,000'



(d) L = 2,500'



(e) L = 3000'



* Minimum % in right lane cannot be less than % of thru traffic in right lane as determined from Fig. 6.

NOTE: These percentages are not necessarily the distributions under free flow or light ramp traffic, but under pressure of high volumes in the right lanes at the point being considered.

Figure 9. Percentage distribution of on- and off-ramp traffic in right lane and auxiliary lane (calculated from Figs. 7 and 8).

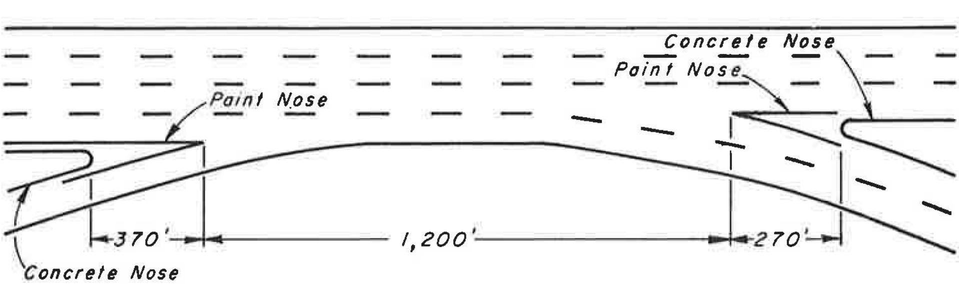


Figure 10.

Because of the length required between entrance and exit ramps, a collector road should be used on all cloverleaf interchanges whenever the weaving volumes exceed 1,200 vehicles an hour. The principle of a cloverleaf with two loops on one side of the freeway is basically incompatible with the principle sometimes expressed as "adequate spacing between interchanges."

If the total distance available for weaving is less than 500 ft, the allowable weaving is less than 2,100 vph. The allowable weaving volume is 1,500 vph when the actual weaving distance is 200 ft. For distances between 200 and 500 ft, the allowable weaving volumes can be assumed to vary linearly.

As an example, in a cloverleaf design where the distance between noses might be 400 ft, the maximum weaving volumes (regardless of the lane distribution factors discussed above) is $1,500 + \frac{200}{300}(2,100 - 1,500) = 1,900$ vph.

SUMMARY

The general procedure for checking weaving and merging capacity is:

1. Establish a given geometric condition.
2. Estimate volumes of the various traffic movements.
3. Use Figures 6 and 9 to determine volume at various check points. At any point in any lane, including the auxiliary lane, the volume should be 1,800 vph or less.
4. Average volume per lane across all lanes should not exceed 1,800 per lane.
5. Number of weaving vehicles in any 500-ft segment should not exceed 2,100 vph. (This ordinarily need not be checked except where the weaving section is 1,000 ft or less.)

If these conditions are met, weaving or merging is workable.

The previous discussion presumes a normal percentage of trucks and relatively level grade. Changes in percentage of trucks or grade will affect the capacities of ramps, and particularly the operational characteristics.

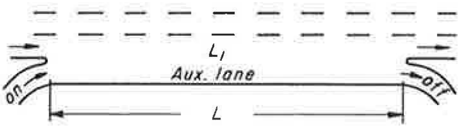
Appendix B gives some examples of the method. The computations can be rather complex in some cases but for general cases figures and tables can be prepared (Figs. 11, 12, and 13).

There are other variables which also affect the critical points on a freeway, but not enough is known about them to incorporate them in the procedure. These variables, which can include alignment, variation in grade, and composition of the traffic, should be considered subjectively in any case. For example, if the procedure shows that a merging lane has a flow rate of about 2,000 vph at some point, but there are very few trucks involved, tangent alignment exists, grade is downhill, or if the number of ramp vehicles is relatively small (500 ±), then perhaps this overloading could be tolerated. On the other hand, if the section is on a plus grade and a curve, then steps probably should be taken to try to reduce the conflict.

As has been noted, 1,800 vph in the right lane or auxiliary lane is below possible capacity and rates of 2,000-2,200 vph have been observed fairly frequently and sometimes operating acceptably. However, there are two reasons for not expecting or designing for this number in all cases:

1. As implied, rates this high are very sensitive to geometric design features and traffic characteristics.
2. Getting such high rates of flow requires that there be no large gaps in the traffic stream. To avoid these gaps (which always occur under free flow conditions), there has to be a constant supply or reservoir of traffic upstream of the merge. Often these extremely high rates are accompanied by some queuing (and thus, stop-and-go driving) upstream of the merge, even though the traffic demand over the short-time period may equal the output at the merge.

General Case - Single lane on- and off-ramp with auxiliary lane.



For L = 1,000'

L_1 thru = 0 ON to OFF = 0		L_1 thru = 300 ON to OFF = 0		L_1 thru = 600 ON to OFF = 0		L_1 thru = 900 ON to OFF = 0	
ON	OFF	ON	OFF	ON	OFF	ON	OFF
0(vph)	1800(vph)	0(vph)	1600(vph)	0(vph)	1250(vph)	0(vph)	950(vph)
900	1800	1100	1600	1100	1250	850	950
1000	1700	1200	1500	1200	1000	900	700
1200	1500	1400	1300	1300	700	1000	400
1400	1300	1500	1200	1400	400	1100	0
1600	1100	1600	900	1500	0		
1800	900	1800	250				
1800	0	1800	0				

L_1 thru = 0 ON to OFF = 200		L_1 thru = 300 ON to OFF = 200		L_1 thru = 600 ON to OFF = 200		L_1 thru = 900 ON to OFF = 200	
0	1600	0	1600	0	1250	0	950
1100	1600	1100	1600	1100	1250	850	950
1200	1500	1500	1200	1200	1000	900	700
1400	1300	1600	900	1300	700	1000	400
1600	1100	1600	0	1400	400	1100	0
1600	0			1500	0		

For L = 1,500'

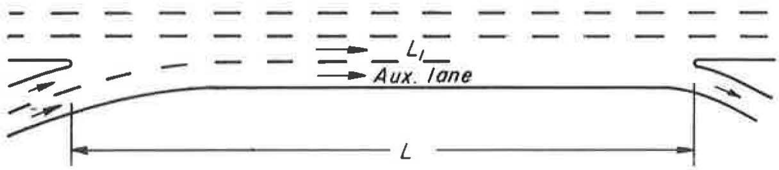
L_1 thru = 0 ON to OFF = 0		L_1 thru = 300 ON to OFF = 0		L_1 thru = 600 ON to OFF = 0		L_1 thru = 900 ON to OFF = 0	
ON	OFF	ON	OFF	ON	OFF	ON	OFF
0	1800	0	1800	0	1500	0	1150
1500	1800	1400	1800	1100	1500	800	1150
1600	1700	1500	1650	1200	1300	1000	800
1800	1500	1600	1450	1400	900	1200	400
1800	0	1700	1250	1600	500	1400	0
		1800	1050	1800	100		
		1800	0	1800	0		

For L = 2,000'

L_1 thru = 0 ON to OFF = 0		L_1 thru = 300 ON to OFF = 0		L_1 thru = 600 ON to OFF = 0		L_1 thru = 900 ON to OFF = 0	
ON	OFF	ON	OFF	ON	OFF	ON	OFF
0	1800	0	1800	0	1800	0	1450
		1750	1800	1200	1800	950	1450
				1400	1400	1000	1250
1800	1800	1800	1650	1600	700	1200	550
		1800	0	1800	100	1400	0
				1800	0		

Figure 11. Acceptable ramp volume rates, calculated from Figure 9 (no more than 1,800 vph at any point in right or auxiliary lane; no more than 2,100 vph weaving in a 500-ft segment).

General Case - 2 lane on-ramp, 1 lane off-ramp, with auxiliary lane.



PRELIMINARY ONLY

For L = 1,500'

L_1 thru = 0 ON to OFF = 0		L_1 thru = 300 ON to OFF = 0		L_1 thru = 600 ON to OFF = 0		L_1 thru = 900 ON to OFF = 0	
ON (vph)	OFF (vph)	ON (vph)	OFF (vph)	ON (vph)	OFF (vph)	ON (vph)	OFF (vph)
0	1800	0	1800	0	1500	0	1100
700	1800	700	1800	1500	1500	1100	1100
2000	1500	1800	1600	1800	0	1400	0
2500	1100	2000	1400				
2700	0	2200	0				

For L = 2,000'

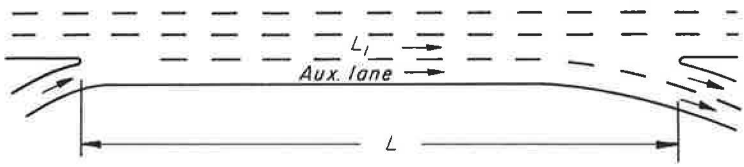
L_1 thru = 0 ON to OFF = 0		L_1 thru = 300 ON to OFF = 0		L_1 thru = 600 ON to OFF = 0		L_1 thru = 900 ON to OFF = 0	
ON (vph)	OFF (vph)	ON (vph)	OFF (vph)	ON (vph)	OFF (vph)	ON (vph)	OFF (vph)
0	1800	0	1800	0	1800	0	1450
2100	1800	1900	1800	1400	1800	1000	1450
2400	1500	2400	0	1900	0	1500	0
2900	0						

For L = 3,000'

L_1 thru = 0 ON to OFF = 0		L_1 thru = 300 ON to OFF = 0		L_1 thru = 600 ON to OFF = 0		L_1 thru = 900 ON to OFF = 0	
ON (vph)	OFF (vph)	ON (vph)	OFF (vph)	ON (vph)	OFF (vph)	ON (vph)	OFF (vph)
0	1800	0	1800	0	1800	0	1800
2800	1800	2400	1800	1800	1800	1100	1800
3600	0	3100	0	2500	0	1800	0

Figure 12. Acceptable ramp volume rates (no more than 1,800 vph at any point in right or auxiliary lane; no more than 2,100 vph weaving in a 500-ft segment).

General Case - 1 lane on-ramp, 2 lane off-ramp, with auxiliary lane.



PRELIMINARY ONLY

For L = 1,500'

L_1 thru = 0, ON to OFF = 0		L_1 thru = 300		L_1 thru = 600		L_1 thru = 900	
ON	OFF	ON	OFF	ON	OFF	ON	OFF
(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)
0	2300	0	1900	0	1500	0	1150
1000	2300	1400	1900	1100	1500	800	1150
1800	1500	1800	1050	1800	100	1400	0
1800	0	1800	0	1800	0		

For L = 2,000'

L_1 thru = 0		L_1 thru = 300, ON to OFF = 0		L_1 thru = 600		L_1 thru = 900	
ON	OFF	ON	OFF	ON	OFF	ON	OFF
(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)
0	2850	0	2400	0	1900	0	1450
1150	2850	1300	2400	1250	1900	950	1450
1800	2300	1700	2050	1800	100	1400	0
1800	0	1800	1650	1800	0		
		1800	0				

For L = 3,000'

L_1 thru = 0, ON to OFF = 0		L_1 thru = 300, ON to OFF = 0		L_1 thru = 600		L_1 thru = 900	
ON	OFF	ON	OFF	ON	OFF	ON	OFF
(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)
0	3600	0	3300	0	3000	0	2700
1100	3300	1000	3000	900	2750	800	2450
1800	3000	1800	2700	1650	2400	1500	2200
1800	0	1800	0	1800	2200	1800	1700
				1800	0	1800	0

Figure 13. Acceptable ramp volumes (no more than 1,800 vph at any point in right or auxiliary lane; no more than 2,100 vph weaving in a 500-ft segment).

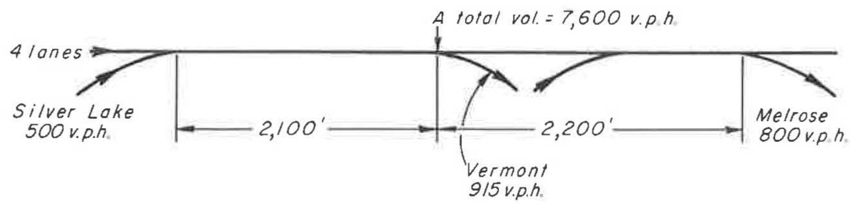
REFERENCES

1. HRB Committee on Highway Capacity, "Highway Capacity Manual." U.S. Government Printing Office, Washington, D.C. (1950).
2. Webb, George, and Moskowitz, Karl, "California Freeway Capacity Study—1956." HRB Proc., 36:587-641 (1957).
3. State of Illinois, City of Chicago, and Cook County Traffic Subcommittee, "Report No. 31 - South Route Expressway Traffic Requirements." Chicago (Sept. 1957).
4. Legarra, J.A., "Progress in Freeway Capacity Studies." American Highway, 27: 3 (July 1958).
5. Keese, C.J., Pinnell, C., and McCasland, W.R., "A Study of Freeway Traffic Operation." HRB Bull. 235, 73-132 (1960).
6. Fukutome, I., and Moskowitz, Karl, "Traffic Behavior and On-Ramp Design." HRB Bull. 235, 38-72 (1960).
7. Newman, L., "Study of Traffic Capacity and Delay at Merge of the North Sacramento and Elvas Freeways." Calif. Highways and Public Works (July-Aug. 1961).
8. Newman, L., "Traffic Operation Study—Whipple Avenue Interchange." Calif. Division of Highways—Unpublished (1962).
9. May, A.D., Jr., "California Freeway Operations Study." (Thompson-Ramo-Wooldridge) (1962).

Appendix A

COMPARISON OF CALCULATED NUMBER OF VEHICLES IN THE RIGHT LANE WITH ACTUAL OBSERVED CASES

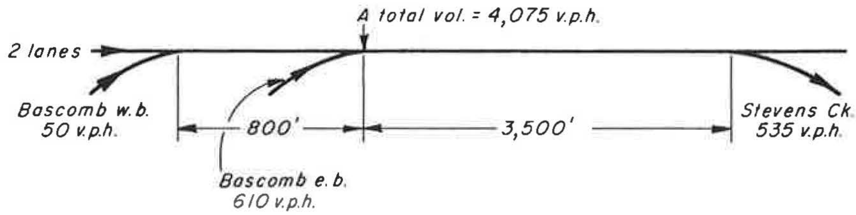
1. Hollywood Freeway at Vermont Avenue.[4]



Thru traffic (not individual in ramp movement with 4,000')
at A = 7,600 - (800 + 915 + 500) = 5,385
% in right lane = 9% = 485
Melrose traffic = 800
% in right lane at A = 60% = 480
Vermont traffic = 915
% in right lane at A = 100% = 915
Silver Lake traffic = 500
% in right lane at A = 18% = 90
Total in right lane at A = 1,970
Actual number observed = 2,003

[4] Data are average of 2 observed peak hours (in 1956—average speed all lanes 45).

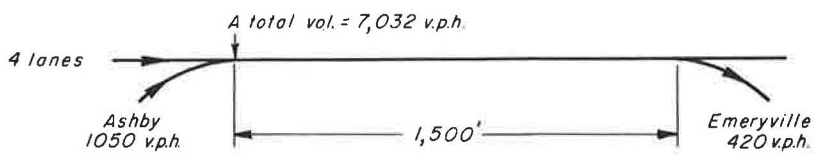
2. Eastshore Freeway at Ashby Avenue.[5]
(Berkeley, California)



Thru traffic (not involved in ramp movement within 4,000')
at A = 7,032 - (1,050 + 420) = 5,562

% in right lane at A = 10%	=	560
Ashby Avenue traffic = 1,050		
% in right lane at A = 100%	=	1,050
Emeryville traffic = 420		
% in right lane at A = 80%	=	340
Total in right lane at A	=	1,950
Actual number observed	=	2,022

3. San Jose-Los Gatos Freeway at Bascom Avenue (San Jose, California).[6]

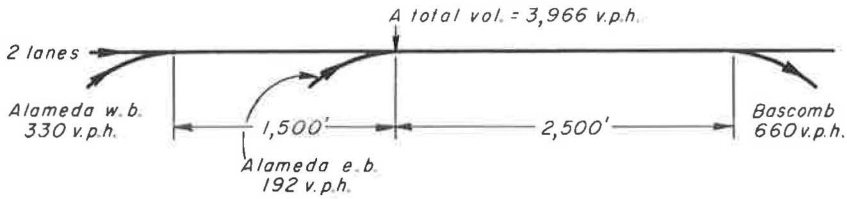


Thru traffic at A = 4,075 - (535 + 610 + 50)
= 2,880
% in right lane = 35%
Stevens Creek traffic = 535
% in right lane = 15% (from Fig. 7) but since the % in the right lane is less than that of the through traffic, this traffic should be assumed to be through traffic (i.e., % of ramp traffic in right lane cannot be less than % of thru traffic in right lane).
Recalculate thru traffic =
2,880 + 535 = 3,415

% in right lane at A = 40%	=	1,370
Bascom Avenue EB traffic = 610		
% in right lane at A = 100%	=	610
Bascom Avenue WB traffic = 50		
% in right lane = 70%	=	35
Total in right lane at A	=	2,015
Actual number observed	=	2,034

[5] Hourly rate for peak 10 minutes during peak hour. Very smooth flow, no queues though there was one merge causing instantaneous stoppage for 15-20 seconds.
[6] Hourly rate for peak 10 minutes. No stoppages occurred during the period.

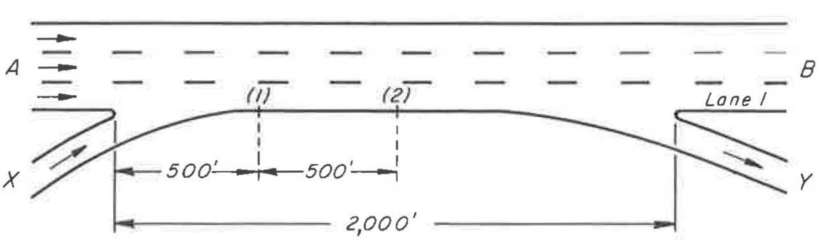
4. San Jose-Los Gatos Freeway at the Alameda (San Jose, California).[7]



Thru traffic at A = 3,966 - (660 + 192 + 330)
= 2,784
% in right lane at A = 35%
Alameda WB traffic = 330
% in right lane at A = 30% (from Fig. 7) but since the % in the right lane is less than that of the thru traffic, this traffic should be assumed to be thru traffic (i.e., % of ramp traffic in right lane cannot be less than % of thru traffic in right lane).
Recalculate thru traffic =
2,784 + 330 = 3,114
% in right lane at A = 40% = 1,245
Alameda EB traffic = 192
% in right lane = 100% = 192
Bascom Avenue traffic = 660
% in right lane at A = 46% = 305
Total in right lane at A = 1,742
Actual number observed = 1,914

Appendix B

Example - 1



Given: (or assumed)

(a) 6-lane freeway
(b) on- and off-ramp 2,000' between concrete noses (no other ramps within 4,000')

[7] Data are hourly rate for peak 10 minutes during peak hour. Operation of merge was very good—capacity was not reached.

(c) Traffic data

A to B	=	4,000
X to B	=	700
A to Y	=	600
X to Y	=	0

Find lane volumes

a. Average lane volume = 5,300 ÷ 3 = 1,770

b. Check lane 1 volume at (1)

Thru traffic in right lane (from Fig. 6) = 0.14 x 4,000	=	560
On-ramp traffic in right lane (Fig. 7 or 9) = 100% x 700	=	700
Off-ramp traffic in right lane (Fig. 7 or 9) = 79% x 600	=	470
Total in right lane at (1)	=	1,730

c. Check lane 1 volume at (2)

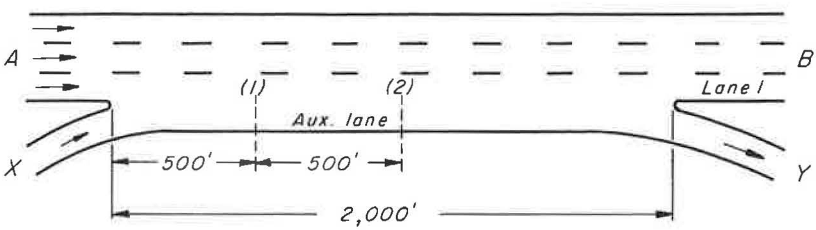
Thru traffic in right lane	=	560
On-ramp traffic in right lane (0.60 x 700)	=	420
Off-ramp traffic in right lane (0.95 x 600)	=	570
		1,550

Comments on the example:

It can be seen that the section would operate satisfactorily and the design would be acceptable since all conditions of the procedure are satisfied. However, a relatively small increase in the volumes or change in traffic patterns could change this fact. It then becomes an economic question whether to build in an extra safety factor by adding an auxiliary lane on this which perhaps might be the most critical section of a freeway. See Example 2 for solution using same volumes with auxiliary lane.

The described procedure only determines whether a certain volume level and traffic pattern will give acceptable operation. It does not evaluate quantitatively how much better operation would be for a certain lower volume level. The method is intended to be used to check a section to insure that it will work and not become a bottleneck for the predicted volume level and traffic patterns.

Example - 2



Traffic:

A to B	=	4,000
X to B	=	1,300
A to Y	=	600
X to Y	=	0

a. Average lane volume at (B) $= \frac{5,300}{3} = 1,770$

b. Check lane 1 volume at (1)

Thru traffic = 0.14 x 4,000 (from Fig. 6)	=	560
On-ramp traffic in right lane = 50% x 1,300 (from Fig. 9)	=	650
Off-ramp traffic in right lane = 0.29 x 600 (from Fig. 9)	=	170
		1,380

c. Check lane 1 volume at (2)

Thru traffic	=	560
On-ramp traffic (0.66 X 1,300)	=	860
Off-ramp traffic (0.19 X 600)	=	110
		1,530

Comments on the example:

As can be seen adding the auxiliary lane greatly increases the ramp capacity.

Usually volumes in the auxiliary lane do not have to be checked unless there is more on-ramp to off-ramp traffic than thru traffic in L_1 which is not likely. Weaving was not checked since the total weave (1,900) is less than 2,100 vph. If it were, however, Figure 9 shows maximum weave takes place in the 1st 500 ft and is 50% of both on-ramp and off-ramp traffic or 950 vph.

Capacities and Characteristics of Ramp-Freeway Connections

JOSEPH W. HESS, Highway Research Engineer, U.S. Bureau of Public Roads, Washington, D.C.

This report presents some of the initial findings of the nationwide Freeway Ramp Capacity Study, sponsored jointly by the Highway Research Board and the U.S. Bureau of Public Roads, for which data were gathered in 1960 and 1961. Additional data recently collected by the Bureau of Public Roads have been incorporated into some of the equations. The capacities associated with ramp-freeway connections are described. These capacities are as follows: for entrance ramps—(1) capacity of the entrance point from the arterial or freeway supplying traffic to the ramp, (2) capacity of the "ramp proper" and (3) capacity of the merging operation at the freeway terminal of the ramp; for exit ramps—(4) capacity of the diverging movement from the freeway to the ramp, (5) capacity of the "ramp proper" and (6) capacity of the ramp terminal connection to the street system.

Any one of these capacities can be the limiting capacity of a ramp. In this study, the emphasis is on the capacity of the merging operation at the freeway terminal of the ramp. In addition, there is some discussion of the capacity of the diverging movement from the freeway to the ramp. Detailed analysis of diverging capacity is under way.

The merging and diverging capacities to and from the freeway not only reflect ramp performance, but also have an important effect on the capacity of the freeway lane. For an entrance ramp, merging capacity is a measure of the ability of the ramp vehicles to make the transition to freeway operation. For an exit ramp, the diverging capacity is a measure of the ability of the freeway vehicles to disengage from the freeway flow and follow their intended path along the ramp.

Before merging capacities can be computed, the freeway lane volume distribution must be known so that lane 1 (i.e., the shoulder lane or righthand lane) volume can be estimated for the given freeway volume. These percentage distributions for four-, six-, and eight-lane freeways are depicted by graphs. As an alternative method in estimating lane 1 volume, equations are presented for use when certain upstream and downstream adjacent ramp conditions are known. These equations make possible an increase in the accuracy of the lane 1 volume calculation. Several of the equations are presented in nomograph form.

Curves are presented showing the free-flowing capacity of various interchange on-ramp connections for different proportions of ramp and lane 1 volumes. In the one group of curves, volume in lane 1 is the independent variable; in the other, volume on the ramp is the independent variable.

Two formulas determined by regression analysis are presented for use in determining free-flow merge capacity at one-lane on-ramps. The formula variables are discussed as to their relative importance and their use is outlined in a sample problem.

Paper sponsored by Committee on Highway Capacity.

Two-lane ramp operations (both on and off types) comprise 42 of the 219 separate studies submitted. These ramps varied widely in both geometrics and traffic characteristics. There were insufficient numbers in any one category to permit derivation of capacity formulas. Several ramp lane distribution curves are shown for some individual two-lane ramp studies. Three of the most interesting two-lane on-ramps are discussed and volumes are quoted. Some general conclusions are drawn from the two-lane ramps submitted.

Finally, several diamond ramps on the Edsel Ford Expressway are offered as representing the type of efficient operation which should be attainable under desirable conditions.

•THE INITIAL concept of a freeway ramp capacity study was developed jointly in August 1958, by O.K. Normann, chairman of the Committee on Highway Capacity and by its Subcommittee on Ramps, under the chairmanship of Leo G. Wilkie. The preliminary study forms were prepared by Mr. Wilkie and presented for consideration at the January 1959 meeting of the committee. The need for a comprehensive picture of ramp-freeway interaction was stressed at that time.

Recognizing this need, the Bureau of Public Roads assumed responsibility for the study. The final layout of field forms and instructions was completed in June 1960. The field phase of the study, carried out by the States and municipal organizations, began shortly thereafter. The Highway Research Board and the Bureau of Public Roads collaborated in bringing the project to the attention of State highway officials and municipal organizations. The data from the first field studies were received in September 1960; as of October 1962, data were received for 219 studies conducted at 195 ramp-freeway connections.

COLLECTION OF DATA

Participating Agencies

The following State highway departments and municipal agencies collected data for this study: California, Colorado, Connecticut, Florida, Georgia, Illinois, Indiana, Kansas, Maryland, Michigan, Minnesota, Missouri, New Jersey, New York, Oregon, Pennsylvania, Rhode Island, Texas, Virginia, District of Columbia, Port of New York Authority, and Cook County (Ill.) Highway Department. Junior engineer trainees of the Bureau of Public Roads studied several locations in Virginia, and eleven locations in Detroit were studied by the author in cooperation with personnel from the Michigan State Highway Department.

Field Procedure

Studies were conducted at both on-ramp and off ramp junctions with freeway, parkway, and expressway facilities. Traffic counts were made by continuous 5-min increments at the nose of the ramp, each observer usually counting one lane of traffic but never more than two lanes. Counts usually began about 30 to 60 min before the peak hour started and continued beyond the peak hour by about the same interval. At on-ramps counts were made at a point just before the nose of the ramp where physical separation still existed between the two flows. At off-ramps, counts were made just downstream from the nose, after physical separation had been established. Figure 1 shows counting locations at both on-ramp and off-ramp locations.

There was considerable variation in speed-recording procedures. Radar speed meters were commonly used, but a number of States used stopwatch time measurements over a measured distance. Several studies were conducted with speeds estimated by observers. Camera and traffic analyzer methods were also used to some extent.

Vehicles were classified as passenger cars or commercial vehicles, the latter including any vehicle with more than four tires.

At each study location an experienced observer kept notes describing within each 5-min counting increment the operation at the study area. Conditions upstream or downstream were also noted, especially when they affected the main study location. The observer's duty was to report any apparent reasons for congested operation although he was cautioned not to speculate. The observer's remarks, along with the recorded speeds, were used as guideposts in identifying the 15-min free-flow periods.

Adjacent ramps, both upstream and downstream, were usually counted simultaneously with the main study location. Remarks on the traffic operation were also made at these adjacent ramps, although freeway lane counts were not taken. At some locations counts were also made at the ramp terminal connection with the local street system. These counts served as a check on the main study area ramp counts and indicated the ability of the discharge point to handle the ramp traffic.

It was decided that continuous counts over 2- or 3-hr periods would be more accurate than short counts interspersed with rest and recording periods which would require interpolation of the data. Because high-volume periods of at least 15 min of free flow were desired, continuous counts and remarks were needed to accurately delineate these periods.

GLOSSARY

The terms used in this report are defined here for ready reference, as follows:

Angle of convergence: The interior angle made between the right edge of lane 1 and the left edge of the ramp at right-hand on-ramps. Where the ramp and/or freeway is on a curve at the nose, a 100-ft chord is drawn back from the nose to its intersection with the inside edge of the ramp and/or freeway lane. The interior angle formed by the chords or by the chord and the tangent edge of the ramp or freeway is then measured as the angle of convergence. The use of the 100-ft chord is an arbitrary choice as an estimate of the average driver's path. The angle of convergence can also be computed if the design radius of the ramp curve and the ramp width at the nose are known. Assuming the design radius given is that for the inner edge of the ramp, the formula would be:

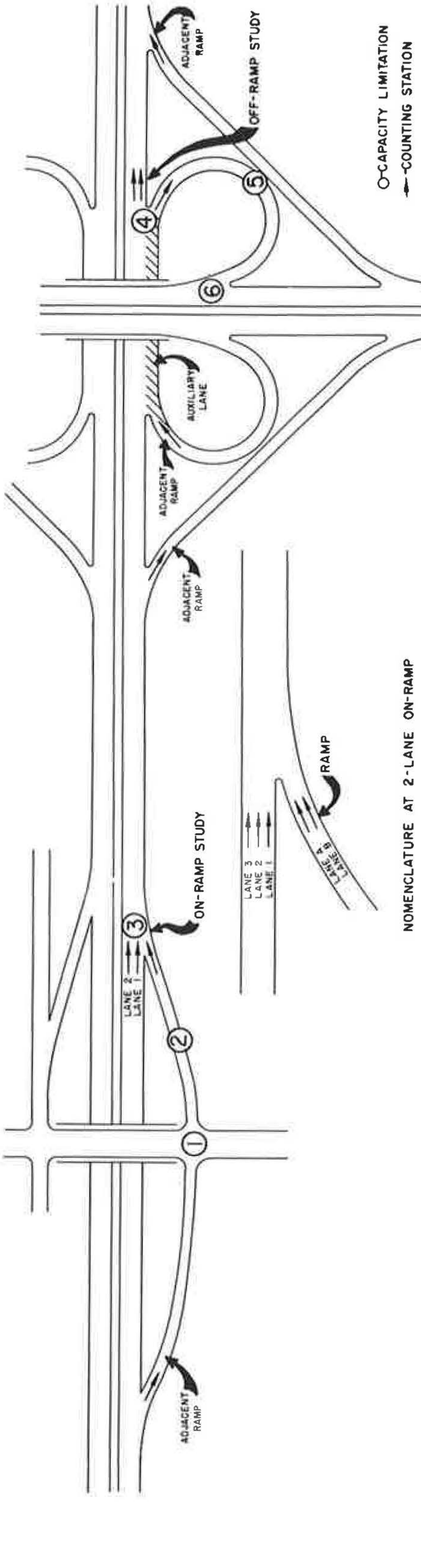


Figure 1. Capacity limitations and counting locations for on-ramp studies.

$$\sin(\text{angle of convergence}) = \frac{50}{\text{Ramp radius (ft)} + \text{Ramp width (ft)}}$$

An alternative would be to consult a table of radii in a surveying textbook, using the chord definition of the radius in getting the degree of curve. The denominator of the above formula would be looked up and the angle of convergence would be ½ D (degree of curve).

Angle of divergence: The interior angle made between the right edge of lane 1 and the left edge of the off-ramp. If either is curved, a 100-ft chord should be employed under the same reasoning as applied to "angle of convergence".

Free-flow merge: Condition where freeway traffic is moving in a uniform manner somewhere in the 35- to 60-mph range. Large fluctuations in speeds are few and traffic is experiencing no conflicts severe enough to cause intermittent braking or congestion. Ramp traffic flow, though possibly slower in speed than the freeway, is continuous without backup on the ramp. The merge of the two streams is normally smooth within the usual adjustments in speed necessary for this maneuver. No specific overall speed should be associated with "free flow," as the design and type of interchange will have an important effect on the speed at any one location. The free-flow periods chosen for this study are of 15-min duration and these volumes are expanded to one hour by multiplying by four (15-min f.f. exp.) The operation during a free-flow period is assumed to be capable of continuance, barring increased demand, backup from downstream, or vehicular accidents. Yet volumes will be in the practical to possible capacity range so that increased demand could cause a breakdown in the operation.

Lane 1: The right-hand lane of the freeway.

Lane 2: The second lane from the right-hand edge of the freeway.

Lane 3: The third lane from the right-hand edge of the freeway.

Median lane: The lane adjacent to the median. In the case of a 6-lane freeway, the median lane would be lane 3.

Peak merge hour: The hour of the highest merge (lane 1 + ramp).

Percent commercial vehicles in merge (% c.v. in merge): The number of commercial vehicles in the merge divided by the total number of vehicles in the merge:

$$\%\ \text{c.v. in merge} = \frac{\text{c.v. (Ramp + Lane 1)}}{\text{Merge volume (Ramp + Lane 1)}} \times 100$$

Percent freeway utilization (% fwy. util.): A measure of the freeway use immediately upstream from the on-ramp nose. It is the hourly freeway volume or 15-min f.f. volume expanded to 1 hour divided by the number of lanes multiplied by 2,000 vph/lane possible capacity per lane:

$$\%\ \text{Fwy. util.} = \frac{\text{Fwy. volume (vph)}}{\text{No. lanes} \times 2{,}000\ \text{vph/lane}} \times 100$$

Ramp lane A: The ramp lane closest to the freeway in the case of a two-lane ramp (see Fig. 1).

Ramp lane B: The ramp lane farthest from the freeway in the case of two-lane ramps (see Fig. 1).

Ramp/merge ratio: A measure of the merge components consisting of

$$\frac{\text{Ramp volume}}{\text{Merge volume (Ramp + Lane 1)}} \times 100$$

Rate of flow or hourly rate: The volume for a short period of time, such as 5 or 15 min, expanded to a vehicles-per-hour figure by the factor

$$\text{Short period volume} \times \frac{60}{\text{Short period (minutes)}}$$

ON-RAMPS

These data were collected under a nationwide "freeway ramp capacity study." In retrospect, this appelation was misleading because it gave the impression that the primary reason for the study was to determine a specific capacity of the "ramp proper." Although this was one objective of the study (if, in fact, such a value can be established), it was a relatively minor objective compared to the need for capacity figures at the merging and diverging ends of the ramp. It was this need which was the primary motivation for this project.

At on-ramps there are the following possible capacity limitation locations (circled numbers, Fig. 1):

1. The entrance point from the arterial or freeway supplying traffic to the ramp.
2. The ramp proper.
3. The merging operation at the freeway terminal of the ramp.

The first of these is outside the scope of this study if the traffic is supplied via a traffic signal system or an ordinary street network. If the ramp traffic is supplied by another freeway or expressway, the "diverging" from that facility is within the scope of this study.

The capacity of the ramp proper is still thought of by some engineers as the limitation of a ramp's ability to carry traffic. In a sense they are right because it is the ultimate capacity limitation, and in a few cases, where there are no limitations to a free flow at either end, this does become the limiting capacity of a ramp. However, conditions at the ramp terminals usually preclude any possibility of obtaining this capacity, making it nearly meaningless from an operational or design standpoint. Unless an additional through-lane is provided beginning at the entrance terminal, an on-ramp is seldom completely loaded with traffic; in most instances where this does occur, it is because the ramp vehicles cannot merge onto the freeway. One of the ramps in this study which did reach the capacity of the "ramp proper" was the cloverleaf inner loop connection from the Long Island Expressway westbound to the Brooklyn-Queens Expressway southbound. This ramp carried 1,918 vehicles in the peak hour, because the capacity restraints at its terminals were removed. At the exit from the Long Island Expressway to the ramp, police directed the outside expressway lane into the ramp. At the other end, the two ramp lanes were necked down to one lane by paint striping before the entrance to the Brooklyn-Queens Expressway. Here, lane 1 of the expressway was coned off, permitting free access for the continuous stream of ramp vehicles. It is reasonable to assume that the volume of traffic handled by this ramp in the peak hour would be considerably less if these unusual steps had not been taken.

The capacity of the merging operation at the freeway terminal of the ramp is most important from the standpoint of the entire freeway system. It is this merging capacity which is examined most thoroughly in this study. Along with weaving, it is one of the most troublesome problems encountered in freeway operation. A faulty merging operation at the freeway terminal of the ramp not only disrupts smooth operation along the freeway but can also cause a backup along the ramp, sometimes extending far enough to block the cross street, frontage road, or lane 1 of an interchanging freeway.

The emphasis in this study is on free-flowing capacity. Perhaps it best corresponds to the 1,500-vph/lane concept of urban "practical capacity." However, the volumes in free-flowing capacity as used in this report ranged up to the values commonly associated with "possible capacity," and in a few instances to less than practical capacity.

A "free flow" is assumed to exist when the following conditions are present: (1) The freeway traffic is moving in a uniform manner somewhere in the 35- to 60-mph speed range; (2) Large fluctuations in speeds are few and traffic is experiencing no conflicts severe enough to cause intermittent braking or wave action; (3) Ramp traffic flow, although possibly slower in speed and more erratic than that on the freeway, is continuous within the ramp demand range without backup on the ramp; and (4) The merging of the two traffic streams is normally smooth within the usual range of adjustments in speed necessary for this maneuver.

No specific overall speed should be associated with free flow, inasmuch as the design and type of interchange has an important effect on the speed range that can be associated with free flow at any one location.

The free-flow periods chosen for this study are of 15-min duration, consisting of three consecutive 5-min counting periods. These 15-min volumes are expanded to 1-hr volume rates by multiplying by four. Hence, the phrase "15-min free-flow expanded period" used throughout this report. The operation during a free-flow period is assumed to be capable of continuance, barring increased demand, backup from downstream, or vehicular accidents. Yet volumes will be in the practical to possible capacity range (urban definition), so that increased demand could cause a breakdown in the operation.

Although there is increasing awareness among those working in the highway capacity field of the need for a peak short-period factor to be applied to design-hour volumes, such a factor was not incorporated in this report for two reasons. First, no nationally applicable procedures for its application have yet been developed. Second, some of the 15-min periods used in this analysis were isolated periods falling outside the actual peak hour, which would have complicated the development of a factor.

Nevertheless, the effect of such a factor, if developed, should be noted. It would allow for the short-term high-volume peak found within the peak hour. The facility would thus be able to continue efficient operation throughout the short-period peak. The short-term peaks as taken from the data reported in this study were higher in the smaller cities (Fig. 2). Free-flow and non-free-flow curves are based on metropolitan area populations. For example, the free-flow curve shows that the 5-min peak volume can be expected to approximate 10.65 percent of the peak-hour volume for a metropolitan area of 250,000, but would only be 9.63 percent for 5,000,000. The standard error on both curves is 8.0 percent above and 7.4 percent below the curve.

If a 6-lane freeway within a metropolitan area of 3,200,000 population carries 5,400 vehicles (1-way) in a peak hour, the 5-min peak volume would be 528 vehicles (9.78% × 5,400) using Figure 2. The hourly rate for the 5 minutes would be 6,336 vehicles (12 × 528). Although it is sometimes possible to sustain this 2,112 vph/lane average for a short period of time on a well-designed freeway, such short-term loads can easily precipitate a stop-and-go type of operation. Application of the factor would reduce the likelihood of this occurring, but would also reduce the design-hour volume. The present AASHO design-hour volume of 1,500 vph/lane for urban freeways was chosen to allow for this short-term peaking.

Maintenance of free-flow operation is not always possible at any selected study point even though the basic ingredients for high-level operation are present. Backups from points downstream can cause congestion for several miles upstream as queueing develops. The ability of the freeway to carry a large volume of traffic past a point is not necessarily hampered by the stop-and-go type operation seen on some freeways during rush hours. Volumes of 1,900 to 2,200 vph/lane are possible, but speeds will be reduced and travel time increased. Higher volumes are often obtained during these congested periods simply because there is a continual steady demand on the facility (Appendix C). There is no need to enumerate the disadvantages of this congested kind of operation which are reflected primarily in traffic delays. The same facility operating with free flow at volumes between practical and possible capacity will still have numerous large relatively open stretches where freeway utilization is low. These open stretches are important, in that they

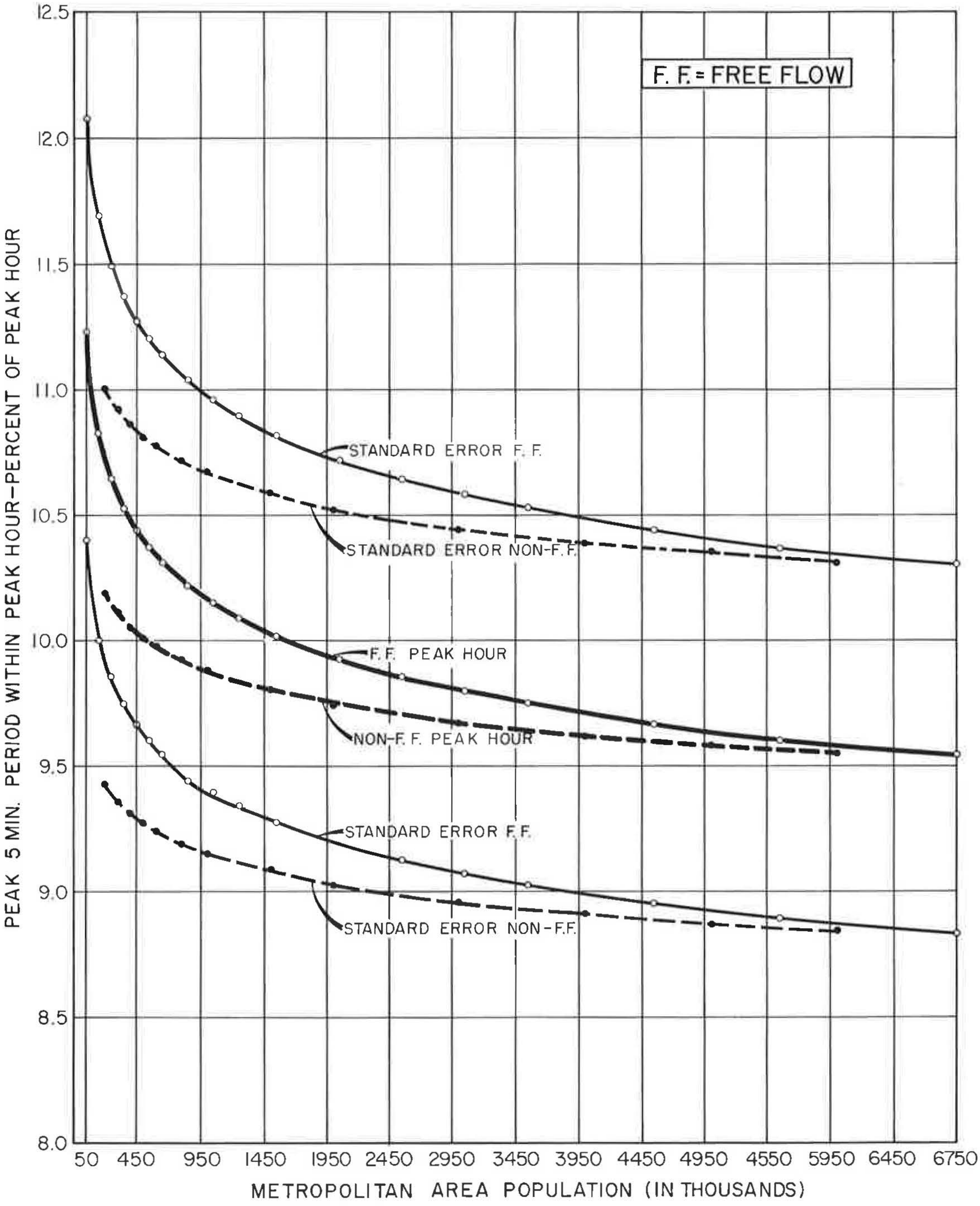


Figure 2. Peaking trends related to population.

allow short periods for recovery of uniform speeds after a momentary surge of ramp traffic has impeded freeway lane 1 speeds. This is especially so at diamond ramps, where platoons of 15 to 20 vehicles are often released by the traffic signal controlling the entrance of traffic from the local street system.

Some of the well-known traffic flow concepts, such as the typical speed/volume relationship, do not always hold true when maneuvering is under way in interchange areas. It is common sense to expect a more erratic operation within complicated interchanges as compared to a simple diamond interchange. City size and driver experience are important in interchange operation: an interchange carrying predominately commuter traffic should have smoother operating characteristics than an interchange geared more toward tourist or interstate traffic.

Poor usage of available speed-change lanes was observed in several studies conducted at ramps serving recreational areas. An example of this was in Michigan at the Kent Lake Road southbound on-ramp to Interstate 96 eastbound. This ramp approximately 20 mi west of Detroit, was studied on a Sunday afternoon when it carries its heaviest volume—Detroiters homeward bound from the Kent Lake recreational area. Many of the 596 peak-hour ramp drivers seemed unaware of the 1,000-ft acceleration lane available for their use, as they either cut directly into lane 1 or stopped at the nose until a suitable gap in traffic appeared. Although the peak-hour freeway volume was only 1,646 vehicles for two lanes, ramp drivers had a difficult time because an entire platoon of vehicles would often be held up by a lead driver who stopped. Granted that the high speed (50- to 60-mph range) of the Interstate 96 vehicles was an inhibiting factor, this facility should still have operated satisfactorily considering the volumes and geometrics. The poor operation seems more a result of driver unfamiliarity with this particular interchange than of overall driver inexperience with interchange driving, because many of these drivers no doubt have had considerable experience on the Detroit expressway system.

Although different lane design volume levels have been established for urban, suburban, and rural locations, there is no easily applied factor to account for driver unfamiliarity. Making the drivers' optimum path readily apparent is always important, but it appears to have even more importance at locations similar to the Kent Lake interchange. Aside from capacity considerations, recognition should be given to the relatively unsafe operation which results when stopped or low-speed ramp vehicles attempt to merge into high-speed through traffic.

Freeway Volume Distribution by Lanes at On-Ramps

The freeway volume distributions by lanes are given in Figures 3, 4, 5, 6, and 7. These freeway volume percentages are as taken just upstream from the ramp nose before merge has taken place. Of course, the volume in lane 1 has a marked effect on the merging operation and the greatest possible accuracy is needed in determining lane 1 volumes at the ramp nose.

For 4-lane freeways, the freeway volume distributions are presented in two groups (Fig. 3)--those at cloverleaf inner loop on-ramps and those at all other types of on-ramps. The reason for this grouping is the difference in operation at cloverleaf interchanges caused by traffic weaving between the adjacent inner loops. In comparison with other types of ramps, the cloverleaf inner loop ramp curves show a heavier use of lane 1 up to freeway volumes of 2,400 vph, despite the loss of lane 1 vehicles at the upstream adjacent outer connection off-ramp. Much of lane 1 traffic is destined for the downstream inner loop off-ramp only 400 to 700 ft away. At freeway volumes above 2,400 vph the comparison shows a heavier use of lane 2 at cloverleaf locations, possibly because drivers wish to avoid the more severe merging and weaving conflicts present at high-volume cloverleaf interchanges.

Three sets of curves are shown for 6-lane freeways at on-ramp locations. Figure 4 contains data for freeways at diamond on-ramp locations only. Figure 5 is derived from 6-lane freeway volume distributions at all types of on-ramps, including diamond ramps but excluding cloverleaf inner loops. Figure 5 also gives the volume distributions where an auxiliary lane is present between the on-ramp and the adjacent downstream off-ramp. Lane 1, where paralleled by an adjacent auxiliary lane starting at

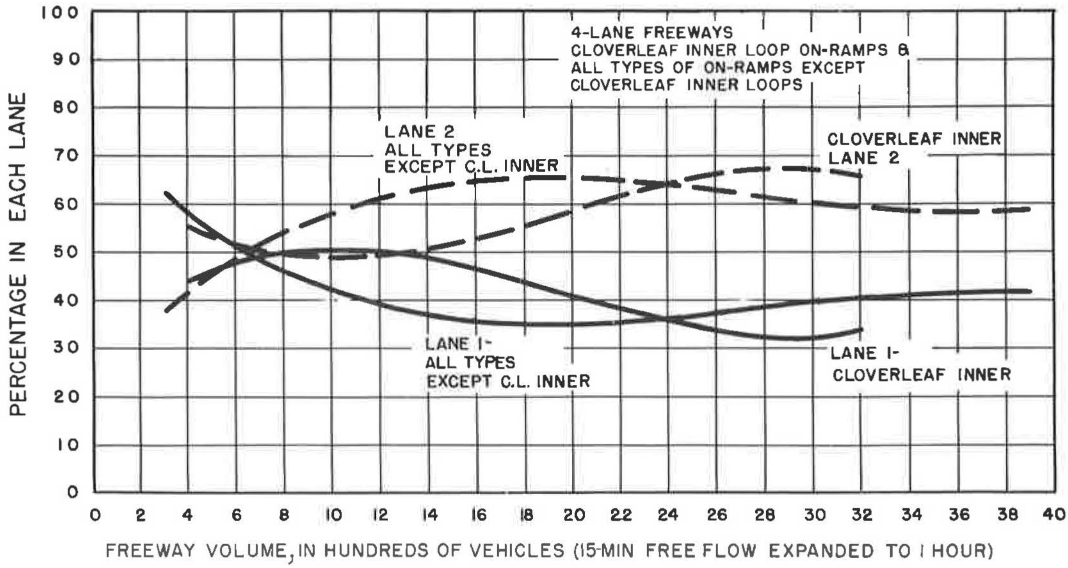


Figure 3. Volume distribution on 4-lane freeways upstream from cloverleaf inner loop and from all other types of on-ramps.

the on-ramp junction, carries approximately eight percentage points more of the freeway traffic as counted at the ramp nose than where there is no auxiliary lane. Investigation disclosed this extra amount approximated the volume exiting at the adjacent downstream ramp at the end of the auxiliary lane. It should be emphasized, however, that the auxiliary lanes contained in the data were of the shorter lengths (none exceeding 1,000 ft) and longer ones might result in a different freeway lane volume distribution. Figure 6 gives lane volume distributions upstream from cloverleaf inner loop on-ramps having auxiliary lane connection with the inner loop off-ramp.

The curves for 8-lane freeways (Fig. 7) should be compared with those for 6-lane freeways. At a given freeway volume the 8-lane freeway will carry less in lane 1 up-

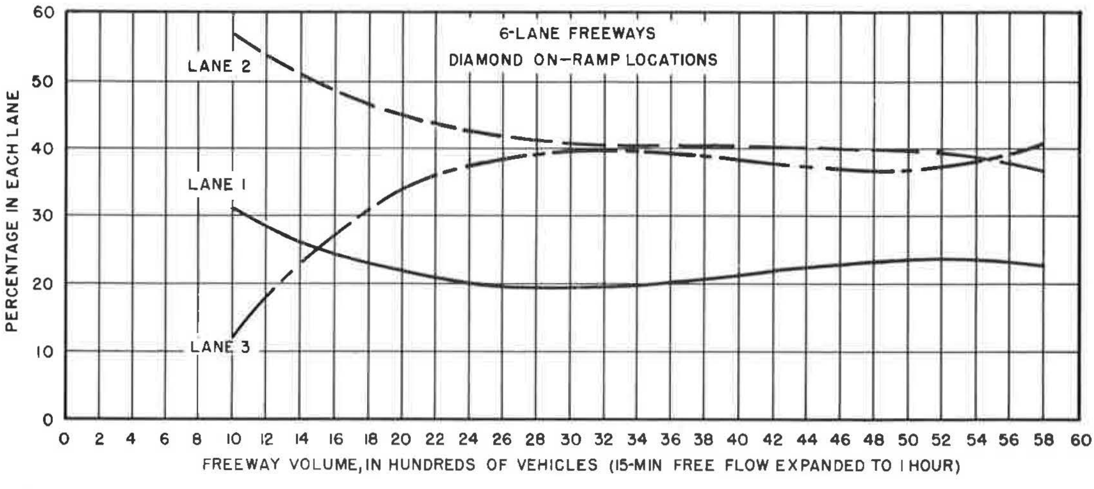


Figure 4. Volume distribution on 6-lane freeways upstream from diamond on-ramps.

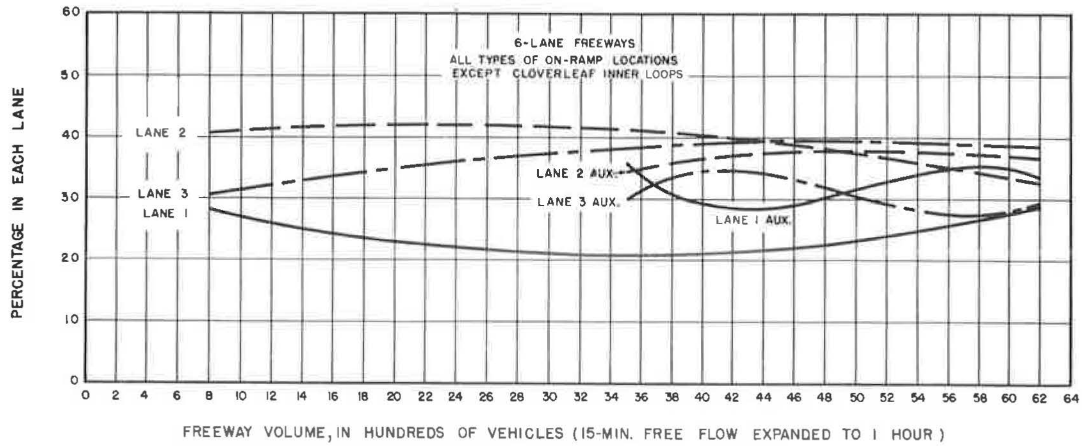


Figure 5. Volume distribution on 6-lane freeways upstream from all types of on-ramps (with and without auxiliary lane at on-ramp entrance) except cloverleaf inner loops.

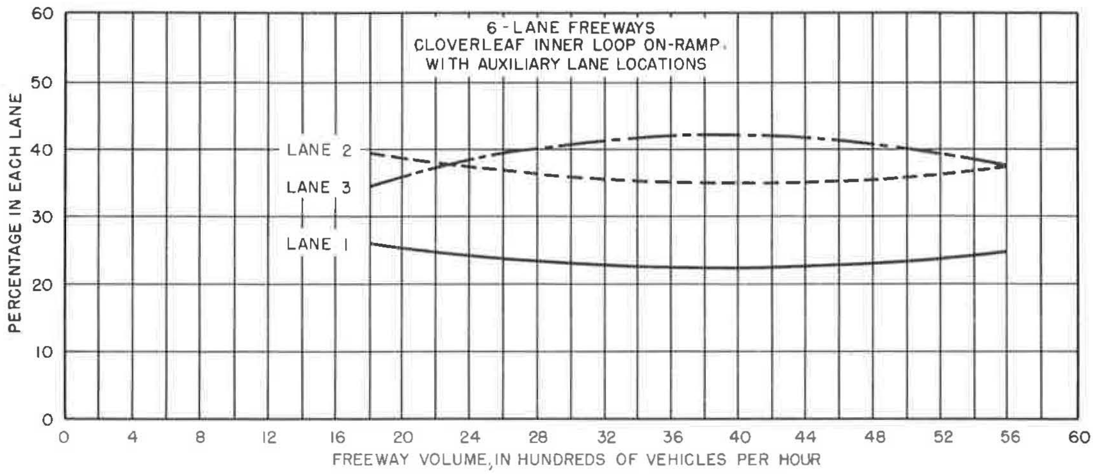


Figure 6. Volume distribution on 6-lane freeways upstream from cloverleaf inner loop on-ramps with auxiliary lane.

stream from the on-ramp because there is an additional high-speed inner lane carrying traffic. Thus, in effect, a higher entrance ramp volume can be accommodated. However, at a given freeway lane volume average, such as 1,500 vph/lane (i.e., 6,000 vph for the 4 lanes of an 8-lane freeway and 4,500 vph for the 3 lanes of a 6-lane freeway), there will be little difference in the lane 1 volume upstream from the entrance ramp and thus little difference in the ramp volume which can merge onto the freeway.

The freeway lane volume distribution at on-ramps varies more at the lower freeway volumes. As an example, data taken at 12 study locations on 6-lane freeways at diamond on-ramps (Fig. 4) in Atlanta, Buffalo, Detroit, and New York City showed the largest residuals in the least squares fit of the lane volume percentage curves at freeway volumes below 2,500 vph. The curves are calculated using data from 41 different 15-min free flow expanded periods in the 1,120- to 5,920-vph freeway volume range. The calculated percentage for several of the low-volume periods differed by

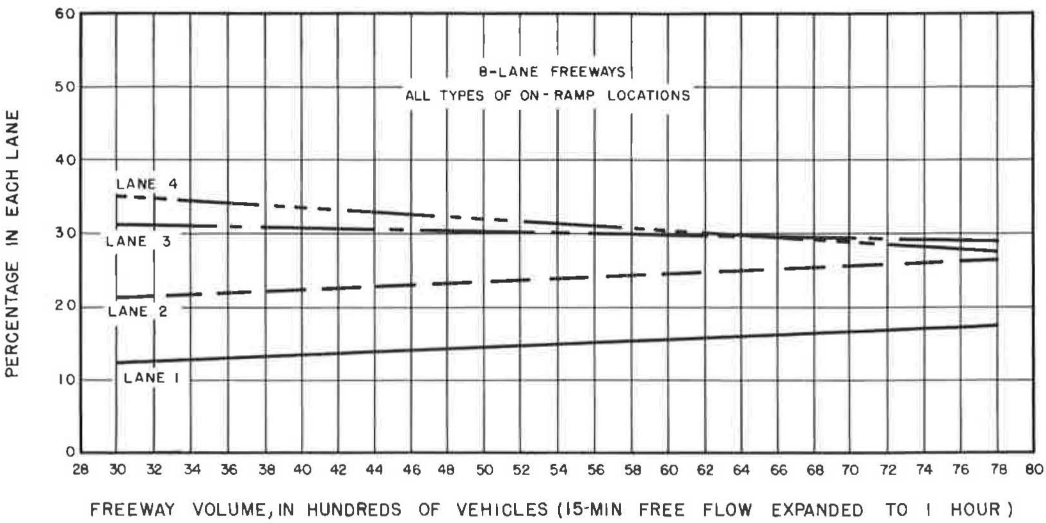


Figure 7. Volume distribution on 8-lane freeways upstream from on-ramps.

as much as 7 to 13 percentage points from the actual percentage. Overall, the standard error of estimate was 2.7 percentage points for lane 1, 3.1 for lane 2, and 3.6 for lane 3.

The behavior of drivers at low freeway volumes cannot be predicted as accurately as at higher volumes because of the great freedom of movement possible. External factors will often influence the choice of a lane for traveling on the freeway. This choice can be exercised at lower volume levels. However, once volumes build up, the individual driver becomes more restricted by his fellow drivers, who are now in closer proximity. The driver's choice of lanes thus becomes more influenced by headways, speeds, and adjacent lane volumes. Once these factors begin to have a pronounced effect on drivers' decisions, there is apt to be less variation in lane percentages between facilities at given volume levels.

The question sometimes arises as to how much effect an on-ramp has on lane 1 traffic. Of course, the lane volume distribution curves do reflect an effect, but what motivates a driver to drive in lane 1 is a question which may never be fully answered. High-volume ramps carrying more than 1,000 vph exert considerable pressure on lane 1 vehicles. Even at more usual ramp volumes, if freeway volumes are light upstream from the ramp there is a tendency for lane 1 vehicles to move over into lane 2 to avoid conflict with the ramp vehicles. This is especially so at low-speed ramp connections, which inhibit through traffic speeds in lane 1. At higher freeway volumes and more usual ramp conditions this tendency is much less pronounced. Commuters' driving habits are fairly well fixed and any tendency to avoid ramp traffic is usually masked because the maneuver to lane 2 or lane 3 may take place well upstream from the ramp. There appears to be a certain amount of local variation in whether drivers move over to avoid ramp vehicles. The degree of conflict the ramp vehicles cause, plus the ease of making a lane change, exert considerable influence on the driver's choice.

Several checks were made on the 6-lane divided Edsel Ford Expressway in Detroit to determine how many cars were moving over within the vicinity of the ramp. In one study, out of 1,003 vph in lane 1, 32 vehicles moved over within the 225-ft stretch upstream from the ramp nose, and 28 others moved over while adjacent to the 575-ft acceleration lane. These 60 vehicles amount to only 6 percent of the lane 1 vehicles moving to lane 2 over the total distance of 800 ft. The ramp volume was 790 vph and the freeway volume 4,372 vph at this location. At another on-ramp in Detroit, 3 percent of 1,426 lane 1 vehicles moved over in the stretch from 100 ft upstream to 300 ft downstream from the ramp nose. The ramp volume was 842 vph and the freeway volume 5,379 vph. It is improbable that all the lane 1 vehicles shifting did so because of the presence of the ramp. Aside from those vehicles which do move over, many lane 1 drivers reflect ramp pressure by edging over near the left edge of their lane at the on-ramp junction. This maneuver gives the ramp vehicle more room laterally in which to jockey while merging. After merging, the majority of the ramp drivers prefer to move over into adjacent lanes as opportunity permits. Studies made recently at four varying freeway sections in Detroit disclosed that approximately 55 percent of the ramp vehicles had moved out of lane 1 within 1-mi downstream from their point of entry onto the freeway. Average freeway lane volumes were in the 1,200- to 2,100-vph range during these studies.

The freeway lane volume distribution curves in this report are least squares fits made without taking into account the variation in ramp volumes. As such, they fairly well represent an average condition. An alternative method for calculating lane 1 volume is given later in this report and in Appendix B. This method takes into account not only the freeway and ramp volumes, but also distances to and volumes of adjacent ramps. Unfortunately, only enough data were available to derive equations for the most usual freeway conditions. Whenever the situation fits within the limits of these formulas, it would increase accuracy to use this alternate method for calculating lane 1 volume rather than the freeway lane volume distribution curves. Nomographs (Figs. 8, 9, and 10) of some of these equations are presented for graphic solution of problems.

Vehicle Storage at On-Ramps

A secondary function performed by ramps is that of providing storage for cars interchanging between facilities. Although engineers endeavor to provide designs that will enable drivers to move without undue delay, traffic volumes often nullify this aim. Lack of adequate capacity at the ramp terminals can force the ramp to function as a storage area for varying periods of time. Stopped or slow-moving vehicles on a ramp are more an irritation than a major operational problem. However, if the available storage cannot absorb the excess demand, there is danger that the backed-up ramp vehicles will block through lanes on the interchanging highway.

As might be expected, interchanges vary considerably in their ability to cope with extreme traffic demands sufficiently well so that congestion is localized and not transmitted via the ramp to the other roadway. Most direct and semi-direct interchanges reported in this study had long ramps, usually two lanes in width, which provided adequate storage when needed. The same was true for the cloverleaf outer connections. Cloverleaf inner loops did suffer from inadequate storage capacity in some of the studies. One ramp which did not suffer from inadequate storage capacity, even though it carried 1,475 vph, was the inner loop from Cross Island Parkway southbound to the Long Island Expressway eastbound on Long Island, N.Y. This well-designed ramp has an auxiliary lane upstream from its exit from Cross Island Parkway and also at its entrance to Long Island Expressway. The ramp, 24 ft wide and fully two lanes operational throughout its length except at its merging end, has a minimum radius of 205 ft with 500-ft radii at its terminals. As shown in Figure 11, the two lanes narrow to one lane 14 ft wide at the merging end of the ramp. Fortunately, from the storage standpoint, if not the travel time standpoint, the ramp is longer than average (1,060 ft). During its peak hour of 1,475 vehicles, this ramp had several 5-min periods when the flow rate exceeded 1,700 vph merging into lane 1 of the Long Island Expressway. The expressway was carrying 3,900 vph in three lanes upstream from the ramp. The substantial storage (running room) afforded by the two ramp lanes localized the congestion which resulted when ramp vehicles were unable to merge fast enough to keep up with the heavy demand. Any design less liberal would have resulted in a backup into Cross Island Parkway, constricting its free-flowing traffic and producing hazardous maneuvers. It was decided to restudy this ramp, concentrating on determination of the number of vehicles traversing the ramp simultaneously and the average speed of the trailing vehicle while the ramp was emptying. During this study the ramp carried a peak of 1,512 vph. The "moving storage" checks, made on the average of once each 5-min period, ranged from 12 to 60 vehicles on the ramp simultaneously. At no time did the ramp vehicles back up into the Cross Island Parkway flow, although several times the

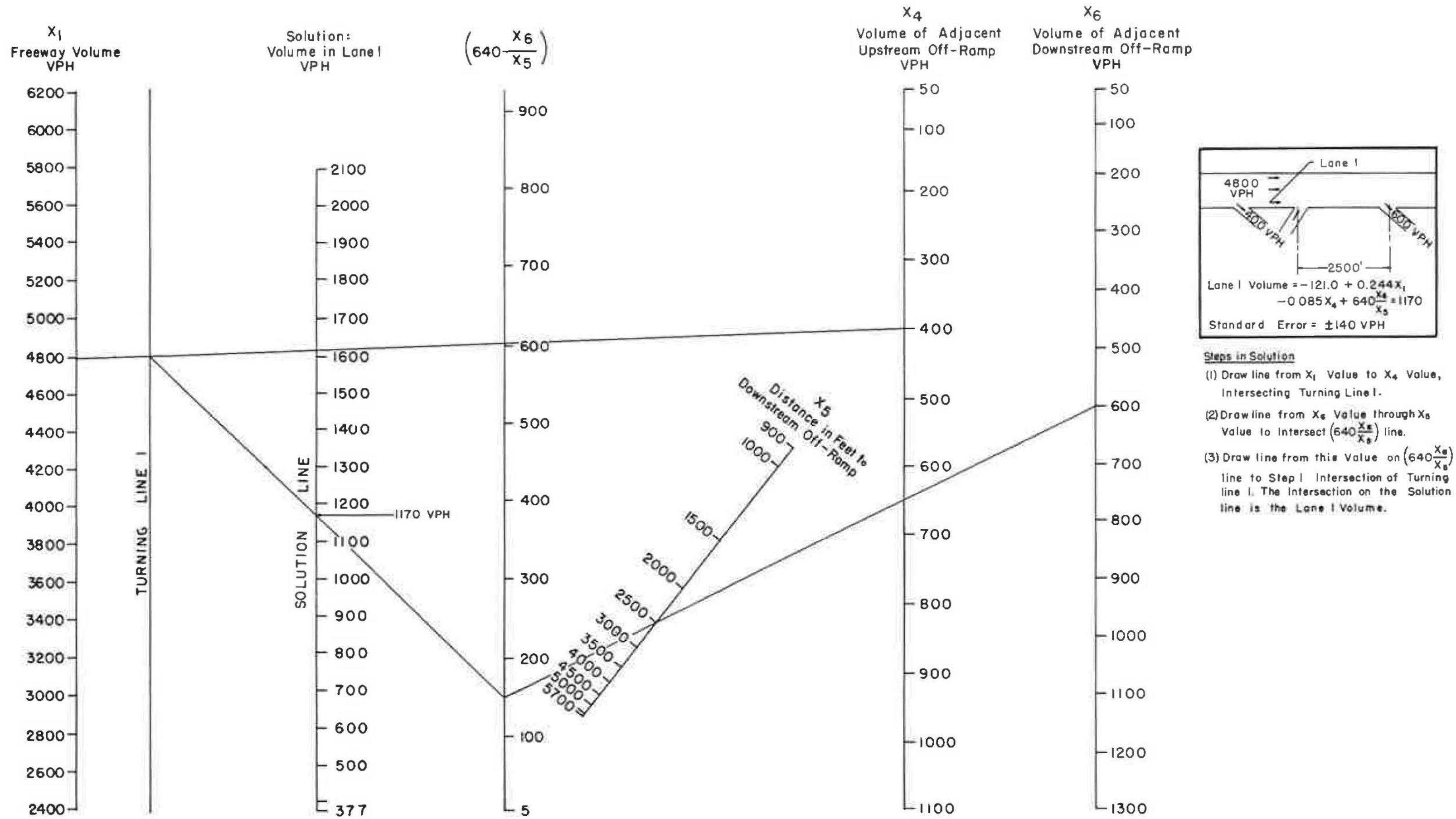


Figure 8. Nomograph for determination of lane 1 volume on 6-lane freeways.

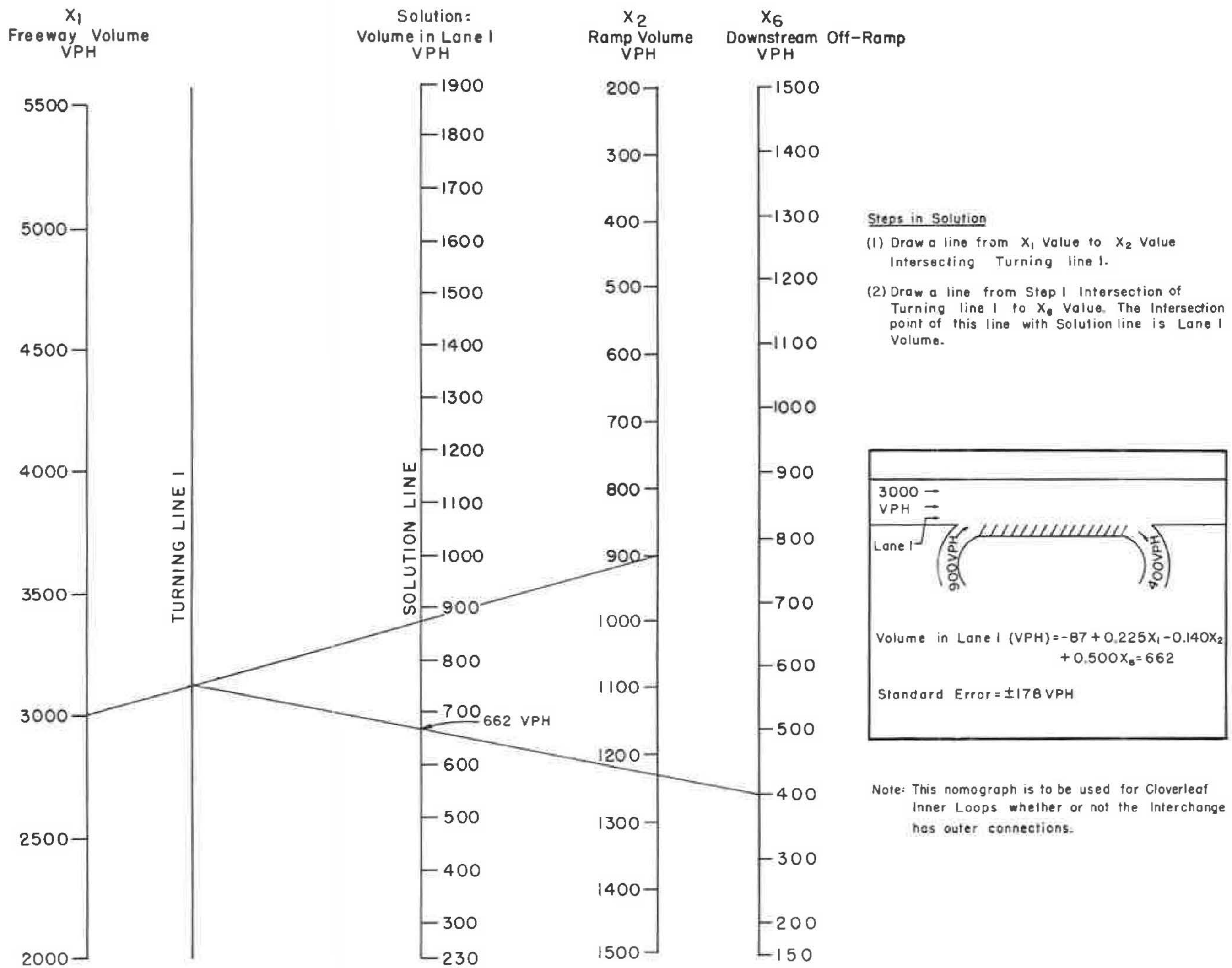


Figure 9. Nomograph for determination of lane 1 volume on 6-lane freeways at cloverleaf inner loops with auxiliary lane.

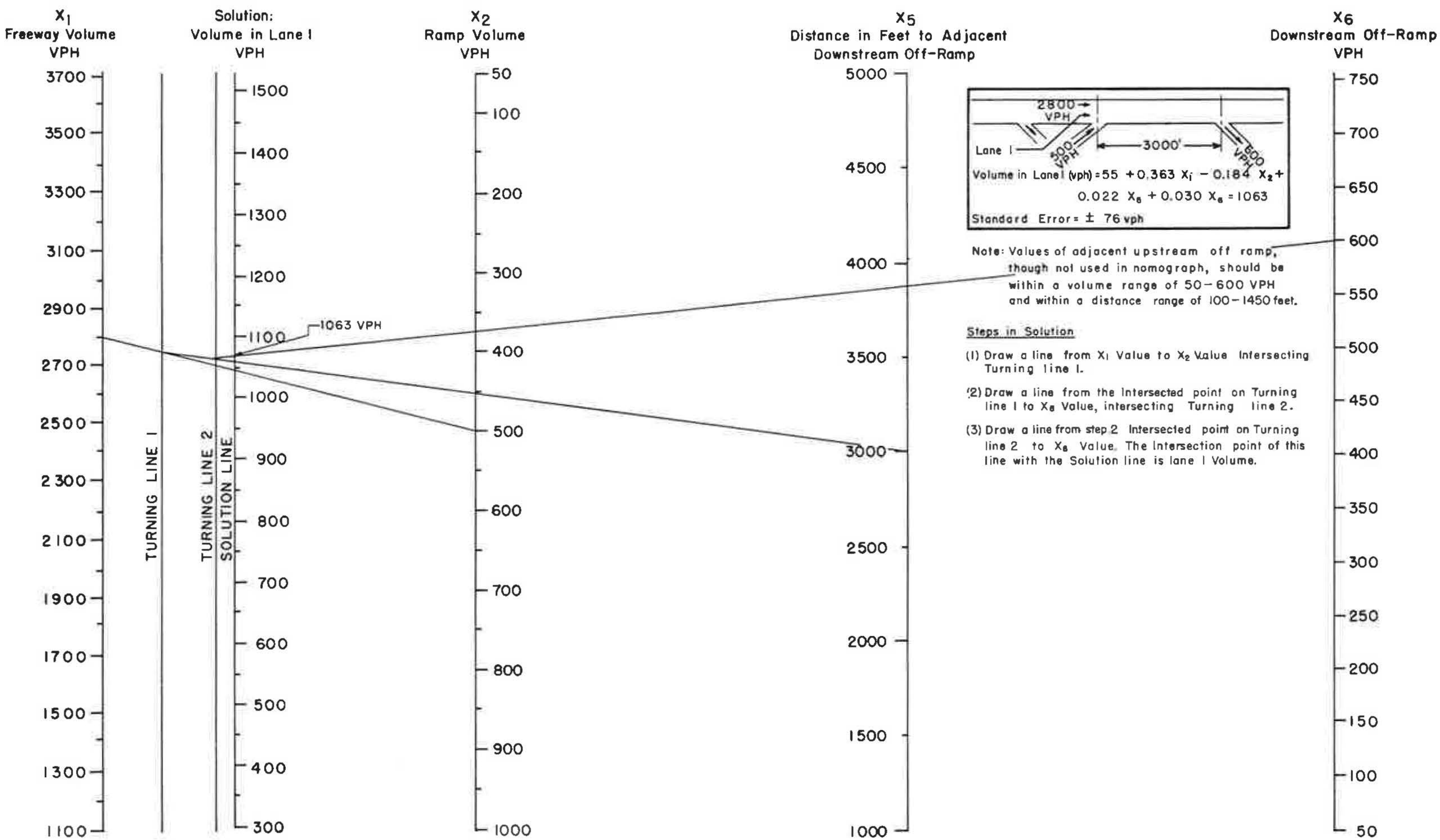


Figure 10. Nomograph for determination of lane 1 volume on 4-lane freeways.

Figure 11. Merging terminal of loop ramp from Cross Island Parkway southbound to Long Island Expressway eastbound on Long Island, N. Y.

ramp was completely full. Average speeds of the trailing vehicles while rounding the ramp were in the 8- to 21-mph range. During 39 min of this hour a vehicle was stalled halfway around the ramp on the inner lane, yet the ramp was able to function successfully during this period at a flow rate of 1,505 vph. The Long Island Expressway, into which the ramp vehicles had to merge, was moving slowly with some stop-and-go operation. The ramp vehicles (1,505-vph rate) had to weave through the off-ramp vehicles (777-vph rate) on the Long Island Expressway over the auxiliary lane distance of 625 ft. Obviously, traffic volumes on the Long Island Expressway were not conducive to free flow on connecting entrance ramps.

Figure 12. Beaubien ramp to Edsel Ford Expressway eastbound, showing poor operation resulting from 1-lane ramp being pressed into 2-lane service by crowding.

Slip ramps, which are usually very short, have a built-in disadvantage, especially where the traffic flow is moderate to heavy on both frontage road and freeway. Any congestion at the merging end can be quickly extended back onto the frontage road lanes.

At diamond on-ramps, storage capacity usually is not of much importance because signalization at the cross street controls the amount of traffic entering the ramp. However, in the case of a short ramp connecting directly to the cross street, vehicle storage can become critical if merging is difficult at the freeway end. This assumes a heavy slug of vehicles released to the ramp by the traffic signal. The data collected at the Beaubien on-ramp to the Edsel Ford Expressway eastbound in Detroit illustrate this situation.

At this location, a 600-ft long, 14-ft

wide (curb to curb) ramp with a 900-ft auxiliary lane operates smoothly at a rate of 900 vph, but when a heavy concentration of rush-hour traffic generated by the dense industrial development nearby delivers 20 to 40 vpm to the ramp, a chaotic situation develops. The stored vehicles on the ramp, waiting to merge, crowd into a two-lane operation with the outside lane using the auxiliary lane while the inside lane is forced to merge directly into an already heavily-loaded lane 1 (Fig. 12). Perhaps this illustrates the advantage of diamond ramps that come off frontage roads, giving the driver the option to continue along the frontage road if the ramp is overtaxed. Ramp vehicles may at times back up onto the frontage road, but this is not as serious as the disruption to the freeway when a one-lane ramp begins to operate as two lanes because of lack of storage room.

Ramp/Lane 1 Volume Proportions for Free-Flow Merge

The volume of traffic which can merge at a ramp-freeway connection is dependent on a number of variables associated with geometrics and traffic characteristics. One of these is the relative proportion of ramp and lane 1 volumes which are combined to make up the merge volume. One cannot expect the ease of merging to remain constant regardless of how the two volumes are distributed. For instance, where 1,600 vehicles must merge and all other variables are held constant, it appears easier to merge 400 ramp vehicles with 1,200 lane 1 vehicles than to merge 800 ramp vehicles with 800 lane 1 vehicles. Also, 1,200 ramp vehicles can usually be merged with 400 lane 1 vehicles more readily than 800 ramp vehicles with 800 lane 1 vehicles. The merge volume in all instances is 1,600 vph but the operation is considerably different in each of the three cases.

Figures 13 and 14 were developed in an effort to determine how the varying combinations of the two flows affect free-flow merge capacity. The curves are the least-squares fittings of ramp and lane 1 volumes for each category of ramp under free-flow merge conditions. The curves are derived from ramps with different geometrics, from freeways both new and old, and from cities of various sizes. No standardization to uniform conditions has been attempted. However, the ramps represented by a specific curve are of a certain category, such as diamonds or cloverleaf inner loops. This grouping provides a measure of uniformity, even though combinations of ramps with different geometrics are necessary to create workable samples. For instance, a diamond ramp having a 700-ft acceleration lane is in the same grouping as a diamond ramp having no acceleration lane. The result is a curve giving a broad average of conditions and reflecting the relative capability of the different type interchanges within these average conditions.

Usually the lane 1 volume is taken as the independent variable in making the least squares calculations, as shown in Figure 13. Strictly speaking, however, lane 1 is not completely independent of the ramp. Rather, the free-flow merge is somewhat of an interaction between the two traffic streams. Then why not use the ramp as the independent variable and lane 1 as the dependent variable? This is done in Figure 14. Arguments can be presented for both cases. The problem is that given the same set of data, the least squares solution of the best curve fit will give different answers for free-flow merge, depending on which flow is taken as the independent variable. As an example, given a cloverleaf outer connection on-ramp with freeway lane 1 volume of 600 vph, how many ramp vehicles can be accommodated while maintaining free-flow merge? Figure 13 shows 800 ramp vehicles to be the answer, whereas Figure 14 would give 1,000 ramp vehicles.

It is the intention in this progress report to show both sets of curves and present the arguments for each. Using lane 1 volume as the independent variable (Fig. 13) seems most logical for several reasons, as follows:

1. Lane 1 generally has the right-of-way.
2. Lane 1 speeds are steadier than those on the ramp.
3. Ramp vehicle drivers generally make the bulk of the merging decisions and the makeup of the lane 1 traffic largely determines how and where the merge is accomplished. Ramp performance appears to be dependent on the lane 1 volume.

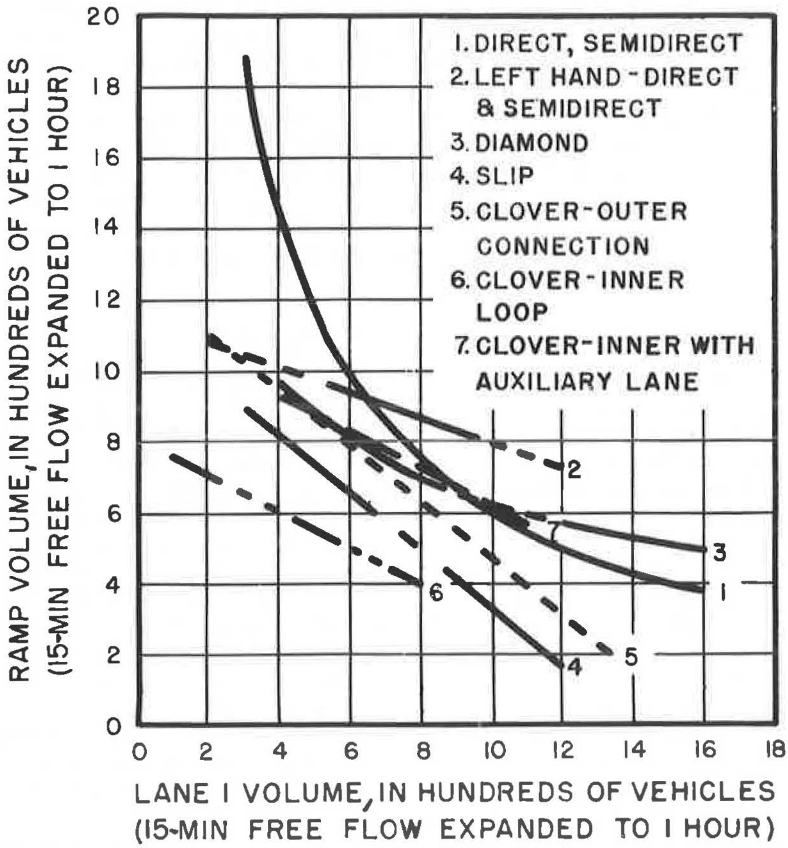


Figure 13. Distribution of ramp and lane 1 volumes for free-flow merge (lane 1 as independent variable).

A less forceful case could be presented for using the ramp as the independent variable (Fig. 14), as follows:

1. The ramp vehicle driver has a maneuver to perform—merging. There is no possible deviation from this goal. The lane 1 vehicle driver, on the other hand, can usually deviate from his path by shifting to lane 2 as a consequence of ramp pressure.
2. High-volume ramps can dominate the merge, forcing lane 1 vehicles to slow down or adjust speeds to those of the merging vehicles.
3. Some lane 1 drivers adjust their speeds to accommodate merging ramp vehicles regardless of the hourly ramp volume or the pressure exerted by the ramp vehicles.

The author prefers the argument in favor of lane 1 as the independent variable for application of the least squares solution. The equations for the curves in Figure 13 are given in Table 1.

As can be ascertained from the standard errors given in Table 1, there is a rather large spread in the data within each ramp category. This is not unexpected because, as mentioned previously, the geometrics and traffic characteristics for ramps within each category varied considerably.

The exponential curves of Figure 13, which are used for direct, semidirect, and diamond ramps, are plotted separately in Figures 15, and 16, together with the upper and lower limits of the standard errors of estimate. The standard error of estimate for an exponential curve is not a constant value, but is a constant percentage above and below the curve value. For instance, the limits of the standard error of estimate shown in Figure 16 for diamond ramps are 34.3 percent above and 25.6 percent below

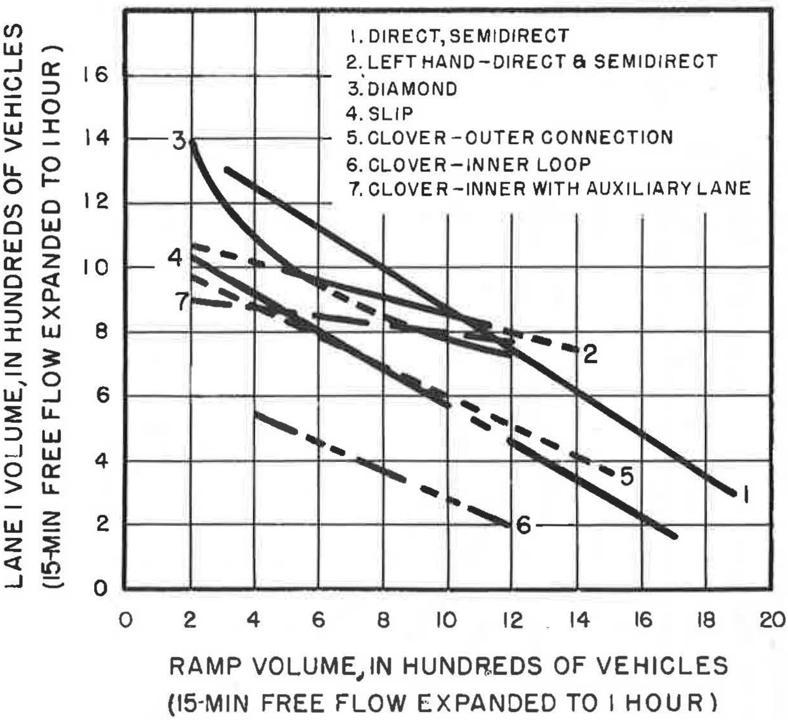


Figure 14. Distribution of ramp and lane 1 volumes for free-flow merge, by ramp type (ramp as independent variable).

the curve value. These percentages apply to the ramp volume, which is added to the given freeway lane 1 volume to form the free-flow merge volume. For a high lane 1 volume combined with a low ramp volume, the possible variation of ramp volume within the standard error of estimate would be quite low. However, for a low lane 1 volume there would be a larger variation in the ramp volume that could be accommodated in the free-flow merge.

Examination of Figure 13 discloses that for most ramp and lane 1 volumes, direct and semidirect ramp connections have the highest free-flow merge volumes, followed in order by diamonds, cloverleaf inner loops with auxiliary lanes, cloverleaf outer

TABLE 1

RAMP VOLUME FORMULAS

Ramp Type	Curve Fit	Equation for Free-Flow Ramp Vol. (vph)	Std. Error of Estimate (vph)
Direct, semidirect	Exponential	$R = 436,909 \text{ (vol. lane 1)}^{-0.955}$	—[a]
Left-hand, direct, semi-direct	Straight line	R = 1,153 - 0.35 (vol. median lane)	336
Diamond	Exponential	$R = 17,029 \text{ (vol. lane 1)}^{-0.479}$	—[b]
Slip	Straight line	R = 1,143 - 0.82 (vol. lane 1)	312
Clover, outer connection	Straight line	R = 1,257 - 0.79 (vol. lane 1)	318
Clover, inner loop	Straight line	R = 805 - 0.51 (vol. lane 1)	273
Clover, inner loop with auxiliary lane	Straight line	R = 1,139 - 0.52 (vol. lane 1)	329

[a] See Figure 15.
[b] See Figure 16.

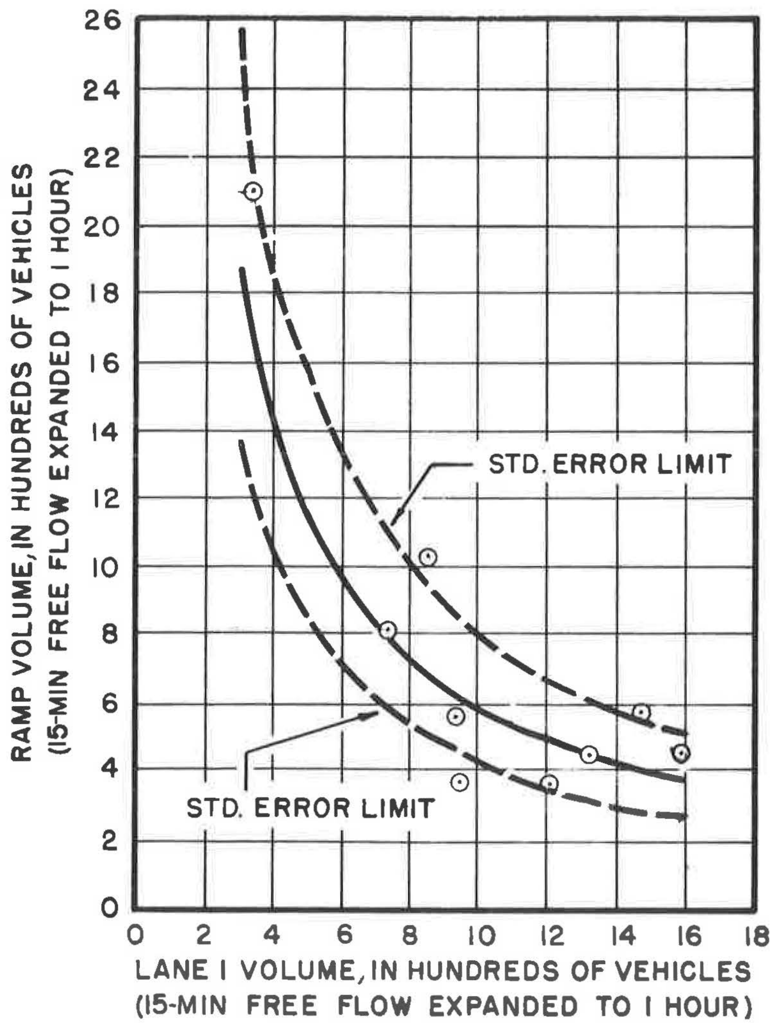


Figure 15. Distribution of ramp and lane 1 volumes for free-flow merge for direct and semidirect ramps (lane 1 as independent variable).

connections, slip ramps, and cloverleaf inner loops without auxiliary lanes. Figure 14 shows much the same order, except that slip ramps and cloverleaf outer connections are nearly identical in merging capacity.

The two sets of curves differ markedly, however, in identifying the optimum proportion for ramp and lane 1 volumes. Figure 13, with lane 1 independent, indicates that the highest free-flow merges can be expected when ramp volumes are low and lane 1 volumes are high, except for direct and semidirect ramps. Figure 14, with the ramp independent, favors high ramp volumes and low lane 1 volumes for highest free-flow merge. This appears logical enough when it is remembered that the ramp is considered independent and at high volumes tends to dominate lane 1 traffic. Finally, the exponential curve shown in Figure 13 and also in Figure 15 for direct and semidirect ramps indicates that when either flow is dominant the free-flow merge volumes will be higher than when the two flows approximate each other in volume. Figures 17 and 18, showing the Route 22 westbound connection to the Garden State Parkway southbound in New Jersey, illustrate a location studied where a heavy ramp (1,800 vph) dominated a merge of 2,100 vph. The location was free flowing, primarily because of the light parkway volume, absence of commercial vehicles, and excellent geometrics. As

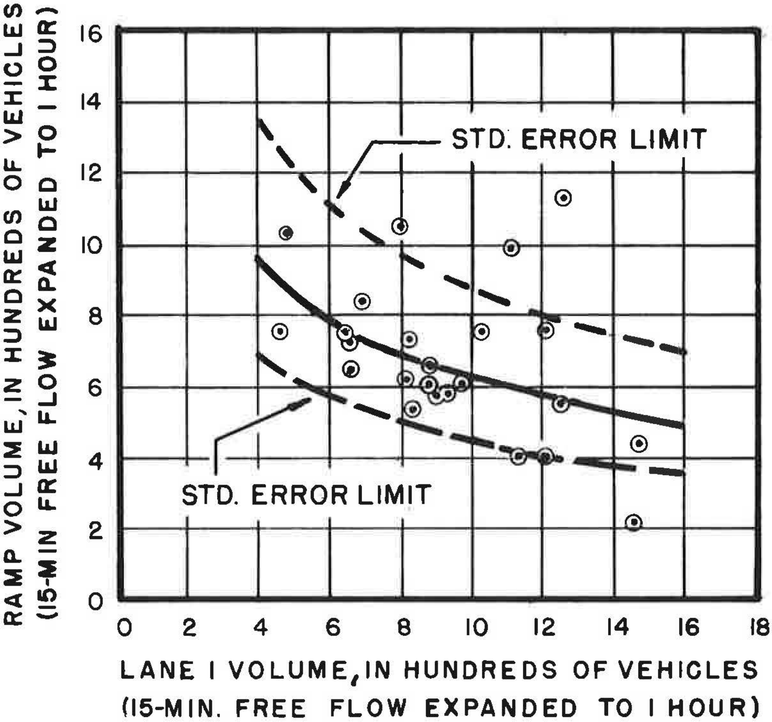


Figure 16. Distribution of ramp and lane 1 volumes for free-flow merge for diamond ramps (lane 1 as independent variable).

Figure 17. Route 22 ramp to Garden State Parkway southbound in New Jersey.

stated previously, the author prefers to treat lane 1 as the independent variable and the ramp as dependent.

When use is made of these curves, lane 1 volume at a given freeway volume can be determined by reference to Figures 3, 4, 5, 6, and 7. The ramp volume determined

Figure 18. Looking back toward merge of Route 22 ramp with Garden State Parkway southbound in New Jersey.

from either Figure 13 or Figure 14 will then represent an average condition for the ramp category. If more accuracy is desired, it would be wiser (though more time-consuming) to apply the regression analysis formulas described in the following section.

Free-Flow Merge for One-Lane On-Ramps

Two formulas were developed by regression analysis for use in computing free-flow merge at 1-lane right-hand on-ramp connections. Appendix A gives a detailed discussion of the variables used and their relative effect on capacity calculations. Table 4 (Appendix A) presents details of the regression analysis.

The first formula, derived from 73 observations at all types of interchanges combined, can be applied to all types of interchanges except left-hand connections. Data were insufficient to permit development of a formula for left-hand ramps. This general formula for one-lane right-hand ramps at all types of interchanges is

$$\text{Free-flow merge (vph)} = 528 + 8.5X_1 - 16.5X_2 + 7.6X_3 - 1.0X_4 + 0.22X_5 + 0.071X_6 \quad \text{(A)}$$

The second formula was derived after deleting the very short ramps and ramps of sharp curvature near the nose (slip ramps and cloverleaf inner loops) from the 73 observations, leaving a remainder of 55 observations. This formula

$$\text{Free-flow merge (vph)} = 441 + 10.0X_1 - 18.0X_2 + 9.5X_3 - 5.0X_4 + 0.014X_5 + 0.068X_6 \quad \text{(B)}$$

should be used only for one-lane right-hand ramps of the following types: Diamond, semidirect, direct, trumpet outer connection, and cloverleaf outer connection. If used erroneously for other types, such as cloverleaf inner loops, it will give values that are too low.

Several of the coefficients in the two formulas differ slightly from the calculated coefficients given in Table 4 (Appendix A). Those differing have been rounded slightly to facilitate computation. This rounding does not affect any free-flow merge computation by more than a few vehicles.

In these formulas the variables are:

X_1 = % freeway utilization. This is a measure of the freeway use immediately upstream from the on-ramp nose. It is the hourly freeway volume (or 15-min free flow expanded to 1 hour) divided by the number of freeway lanes multiplied by 2,000 vph possible capacity per lane, or

$$\% \text{ Freeway utilization} = \frac{\text{Freeway volume (vph)}}{\text{No. of lanes} \times 2,000 \text{ vph/lane}} \times 100 \qquad \text{(C)}$$

X_2 = % commercial vehicles in the merge. This is the number of commercial vehicles in the merge (ramp + lane 1) divided by the expected number of vehicles in the merge, or

$$\% \text{ c.v. in merge} = \frac{\text{c.v. (Ramp + Lane 1) vph}}{\text{Expected merge volume (vph)}} \times 100 \qquad \text{(D)}$$

X_3 = ramp/merge ratio. This is a measure of the merge components, consisting of the ramp volume divided by the merge volume, or

$$\text{Ramp/Merge ratio} = \frac{\text{Ramp volume (vph)}}{\text{Expected merge (Ramp + Lane 1) volume (vph)}} \times 100 \qquad \text{(E)}$$

X_4 = angle of convergence, in degrees. This is the interior angle made between the right edge of lane 1 and the left edge of the ramp at right-hand on-ramps. (The glossary gives details on measuring this angle when the ramp and/or freeway is curved.)

X_5 = length of acceleration lane, in feet.

X_6 = metropolitan area population, in 1,000's. (This value should not exceed 5,000 as applied to the formula.)

In using the formulas, whole numbers and not decimal equivalents should be used for the percentages expressed in X_1, X_2, and X_3 (i.e., for 27 percent use 27, not 0.27). The metropolitan area population, X_6, should be obtained from the 1960 census, keeping in mind that for metropolitan area populations larger than 5,000,000, the figure 5,000 should be used.

The results of the two formulas, broken down by ramp types, are compared in Table 2. The formula based on 73 observations generally predicts higher values for cloverleaf outer connections and lower values for diamond ramps than does the other. All other ramp types are grouped together, with similar results for the two formulas. Everthing considered, any difference in results between the two formulas is minor and for simplicity the formulas based on 73 observations should be used. The other formula, although more limited as the ramp types represented, has the advantage of a lower standard error of estimate.

It is interesting to note that in Table 4 (Appendix A) the mean value of the free-flow merge is 1,569 vph for the 73 observations. This is a close approximation of the 1,500 vph/lane assigned as urban practical capacity for freeways. The standard deviation of 288 vph indicates quite well that there is no magic number which can be used as

TABLE 2

COMPARISON OF RESULTS FROM FORMULAS BASED ON 73 AND 55 OBSERVATIONS

Ramp Type	Observations	Free-Flow Merge (vph)			Difference in Means[a]
		True Mean	Predicted Mean		
			73 Obs.	55 Obs.	
Clover, outer	17	1,502	1,545	1,503	+42
Diamond	27	1,645	1,621	1,655	-34
Direct; semidirect; and trumpet, outer	11	1,724	1,684	1,695	- 9

[a]73 observations formula compared with 55 observation formula.

a merge capacity figure for general application. Of the 73 observations, 51 (70%) fall within the 1,281- to 1,857-vph range of one standard deviation. This appears to be a rather large range but, once again, it should be remembered that the data used are not only from different sections of the nation but also represent a wide range of design and traffic conditions. The variation in merge capacity within a given more or less homogeneous system should be considerably less.

Also of interest are the means of the freeway volumes upstream and downstream from the ramp for the 73 observations. Upstream from the ramp (before merge) the mean of the freeway volumes was 9.5 percent below practical capacity. The mean of the ramp volumes was 597 vph. Downstream (after merge) the mean of the freeway volumes was 4.8 percent above practical capacity.

Figure 19 presents a means of determining the percentage of commercial vehicles in lane 1 of the freeway at the ramp nose. A word of caution is needed regarding its use. Partially because of local laws, there is much variation between cities in the truck distribution among lanes. If local data are available, it would increase the accuracy to use them rather than Figure 19 when applying the formulas.

The formulas can be used for a number of purposes. Used in conjunction with lane volume distribution curves or lane 1 volume equations and commercial vehicle distribution (Fig. 19), the formulas provide a much needed capacity computation tool. Also, if some traffic counts are made, facilities already in operation can be evaluated for quality of performance. The formulas could be very useful where possible ramp closures or monitoring are being evaluated in the hope of maintaining a free-flow volume level on congested freeways. Another possible use is the prediction of future trouble spots as traffic volumes increase on newly opened networks.

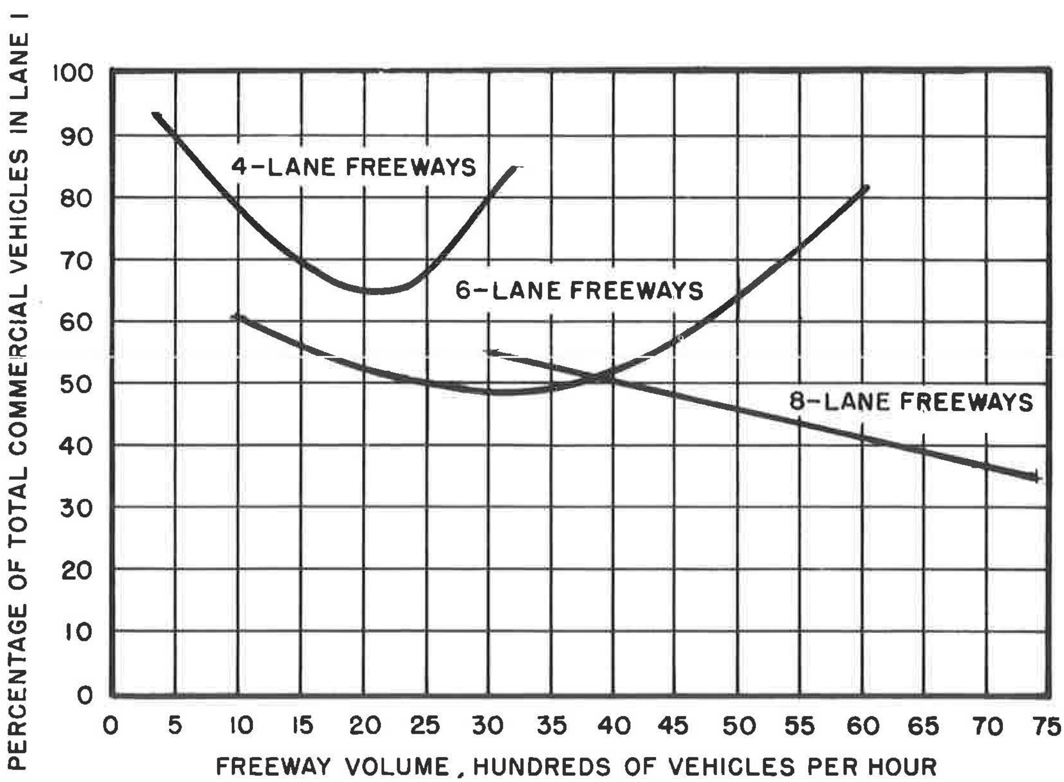


Figure 19. Percentage of total commercial vehicles in lane 1 of 4-, 6-, and 8-lane freeways immediately upstream from on-ramp entrances.

Sample Problem No. 1

Given: A semidirect interchange along a 6-lane freeway in Detroit, Mich. The freeway carries 4,300 vph just upstream from the ramp nose. Ramp volume is 800 vph. Commercial vehicles make up 3 percent of the freeway traffic and 3 percent of the ramp traffic. The angle of convergence is 10° and the length of acceleration lane is 600 ft.

Find:

1. The "expected merge" for this on-ramp connection under the given traffic volumes.
2. The predicted "f.f. merge" using the formula derived from 55 observations.
3. Adequacy of design without considering the standard error of estimate.
4. Adequacy of design considering the standard error of estimate given in Table 4.

Solution:

Using Figure 5, lane distribution for 6-lane freeways, 21.5 percent of the freeway stream will be in lane 1, and land 1 (vph) = 0.215 × 4,300 = 925.

"Expected merge" = 925 vph (lane 1) + 800 vph (ramp) = 1,725 vph.

Number of commercial vehicles in freeway stream = 0.03 × 4,300 = 129.

Number of commercial vehicles in ramp traffic = 0.03 × 800 = 24.

Using Figure 19 for commercial vehicle distribution, 55 percent of the 129 freeway commercial vehicles will be in lane 1, or commercial vehicles in lane 1 = 0.55 × 129 = 71.

Using the 55-obs. formula:

Plus quantities for formula

Constant = 441

X_1, % fwy. util. $= \dfrac{4,300}{3 \times 2,000 \text{ vph/lane}} \times 100 = 72$

X_3, ramp/merge ratio $= \dfrac{800}{925 + 800} \times 100 = 46$

X_5, acceleration lane = 600

X_6, metropolitan area pop. = 3,762 (from 1960 census) (1,000's)

Minus quantities for formula

X_2, % c.v. in merge $= \dfrac{71 + 24}{925 + 800} \times 100 = 5.5$

X_4, angle of convergence = 10°

Applying the 55 obs. formula:

$$\text{Free-flow merge} = 444 + 10.0(72) - 18.0(5.5) + 9.5(46) - 5.0(10) + 0.14(600) + 0.068\,(3,762) = 1,789 \text{ vph.}$$

Inasmuch as the 1,789-vph free-flow merge predicted by the formula is more than the "expected merge" of 1,725 vph, the facility can be considered adequate. However, if the design standard is to keep the free-flow merge within the standard error of estimate, the designer would have to consider the minimum free-flow merge within the standard error of estimate, which is 1,630 vph (1,789 vph prediction - 159 vph standard error of estimate). The probability of having a free-flow merge of more than 1,630 vph is approximately 0.84. The 1,630 vph is less than the "expected merge," so adjustments would have to be made in the design.

Answers:

1. 1,725 vph "expected merge."

2. 1,789 vph predicted free-flow merge.
3. Design is adequate without taking into consideration the standard error of estimate.
4. Design is inadequate if standard error of estimate is applied.

Those using these formulas should understand the relationship between the "expected merge" and the "free-flow merge." The "expected merge" is the merge which is forecast, taking into account the ramp and freeway volumes. If it is less than the computed "free-flow merge," the facility will operate satisfactorily and the merge taking place will be the "expected merge." However, if the "expected merge" is higher than the "free-flow merge," the indication is that the operation will be congested because more vehicles will be attempting to merge than can be accommodated in a satisfactory manner by the facility.

By itself, the free-flow merge volume has limited significance. The facility can be operating very well with a low free-flow merge volume, provided the "expected merge" is even lower. Increasing the freeway and/or ramp volumes would increase the computed "free-flow merge" but the "expected merge" would increase even more rapidly so that congested operation would soon result.

Different "expected merge" and "free-flow merge" volumes will result if the proportion of the ramp and freeway volumes are varied while keeping a constant total volume. For instance, using the sample problem condition and varying the volumes up and down by 200-vph increments so that computations are made for 600 ramp vehicles merging with 4,500 freeway vehicles and 1,000 ramp vehicles merging with 4,100 freeway vehicles, the results are as follows:

Free-Flow Volume (vph)				
Total	Freeway	Ramp	Expected Merge	Predicted Free-Flow Merge
5,100	4,100	1,000	1,861	1,835[a]
5,100	4,300	800	1,725	1,789
5,100	4,500	600	1,568	1,734

[a]Congestion predicted.

The foregoing comparison shows that as the ramp volume increases, the "expected merge" and the "free-flow merge" volumes both increase, but the former much more rapidly. In the case of 1,000 ramp vehicles merging into a freeway stream of 4,100 vehicles, some congestion can be expected because the "free-flow merge" is less than the "expected merge." Much as experience with freeway operation might lead one to assume, the best operation of the three cases cited is when 600 ramp vehicles merge into a freeway stream of 4,500 vehicles.

Alternative Method for Computation of Free-Flow Merge

The formulas used in the preceding section depended on a computation of lane 1 volume by the use of curves set up for varying freeway volumes (Figs. 3, 4, 5, 6, 7). These least squares curves represent the best fit for the lane use data obtained in this study. Indirectly, the curves reflect study ramp pressure on lane 1 volumes, adjacent ramp action on lane 1 volumes, and the effects of the various other components (such as signing and geographical location) which are determinants in the use of lane 1. These curves are more accurate at high freeway volumes than at volumes below practical capacity. At these lower volume levels there is more margin for error as local conditions (such as location and volume of adjacent ramps) exert more of an influence on the freeway volume distribution.

In an attempt to more closely fit the conditions at hand and narrow the margin of error, five equations have been developed by multiple regression analysis for use in calculating the lane 1 volumes used in the free-flow merge calculations. These equations take into account the distances to and volumes of adjacent upstream and downstream off-ramps, as well as the freeway and ramp volumes at the connection for which computations are being made. Unfortunately, there were insufficient data to permit derivation of equations for conditions other than the most common possibilities. Table 3, as well as Appendix B, gives the conditions for which the equations are applicable. Those using the equations should not extrapolate or use values outside the ranges shown in Table 3. For those who prefer graphic solutions, nomographs (Figs. 8, 9, and 10) are given for Eqs. 1, 4, and 5.

The equations and the broad requirements for the use of each are as follows:

Equation No. 1—

Condition: For 6-lane freeways when the on-ramp under consideration is bracketed by adjacent off-ramps, upstream and downstream, and no auxiliary lane connection exists to the adjacent downstream off-ramp.

$$\text{Volume in lane 1 (vph)} = -121 + 0.244\,(\text{freeway volume in vph}) - 0.085\,(\text{volume of adjacent upstream off-ramp in vph}) + 640\,\frac{(\text{volume of adjacent downstream off-ramp in vph})}{(\text{distance, in feet, to adjacent downstream off-ramp})}$$

Equation No. 2—

Condition: For 6-lane freeways when the on-ramp under consideration has an adjacent upstream off-ramp and is connected to an adjacent off-ramp less than 1,000 ft downstream by an auxiliary lane.

$$\text{Volume in lane 1 (vph)} = 62 + 0.232\,(\text{freeway volume in vph}) - 0.072\,(\text{ramp volume in vph}) - 0.041\,(\text{length, in feet, of auxiliary lane}) + 0.432\,(\text{volume of adjacent downstream off-ramp in vph})$$

Equation No. 3—

Condition: For 6-lane freeways when the on-ramp under consideration is connected to an adjacent off-ramp less than 1,000 ft downstream by an auxiliary lane, and there is either no nearby upstream ramp or, if so, its volume is negligible.

$$\text{Volume in lane 1 (vph)} = -162 + 0.273\,(\text{freeway volume in vph}) - 0.195\,(\text{ramp volume in vph}) + 0.635\,(\text{volume of adjacent downstream off-ramp in vph})$$

Equation No. 4—

Condition: For 6-lane freeways at cloverleaf interchanges where the inner loop on-ramps is connected to the inner loop off-ramp by an auxiliary lane. The interchange may or may not have outer connections. The equation does not require them and applies only to inner loop on-ramps.

$$\text{Volume in lane 1 (vph)} = -87 + 0.225\,(\text{freeway volume in vph}) - 0.140\,(\text{ramp volume in vph}) + 0.500\,(\text{volume of adjacent downstream inner loop off-ramp in vph})$$

Equation No. 5—

Condition: For 4-lane freeways when the on-ramp under consideration is bracketed by adjacent off-ramps, upstream and downstream, and no auxiliary lane connection exists to the adjacent downstream off-ramp.

$$\text{Volume in lane 1 (vph)} = 55 + 0.363\,(\text{freeway volume in vph}) - 0.184\,(\text{ramp volume in vph}) + 0.022\,(\text{distance in feet to adjacent downstream off-ramp}) + 0.030\,(\text{volume of adjacent downstream off-ramp in vph})$$

Sketches of these layouts are shown in Table 5 (Appendix B), together with the equations and associated statistical data.

TABLE 3

REQUIRED RANGES OF VARIABLES FOR VALID USE OF EQUATIONS

Eq.	Volume (vph)		Adj. Upstream Off-Ramp		Adj. Downstream Off-Ramp	
	Freeway	Ramp	Distance (ft)	Volume (vph)	Distance (ft)	Volume (vph)
1	2,400 - 6,200	100 - 1,700	900 - 2,600	50 - 1,100	900 - 5,700	50 - 1,300
2	1,900 - 6,200	150 - 1,900	450 - 2,150	50 - 1,000	550 - 950	50 - 1,000
3	1,900 - 6,200	50 - 1,900	-	-	550 - 950	50 - 1,000
4	2,000 - 5,600	200 - 1,500	-	-	450 - 850	150 - 1,500
5	1,100 - 3,700	50 - 1,000	100 - 1,450	50 - 600	1,000 - 5,000	50 - 750

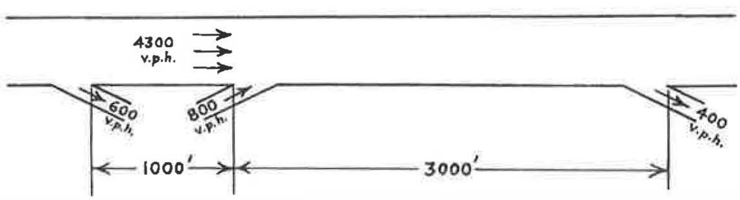
Those who use the equations should not be confused because some of the required validating conditions are not found as variables in the equations. For example, in Eq. 2 an adjacent off-ramp is required between 450 and 2,150 ft upstream from the ramp under consideration. However, the equation does not contain variables relating to this specified adjacent upstream off-ramp. These variables are missing from the equation because their effect was found to be negligible and so their input was deleted in the derivation of the formula. Nevertheless, conditions other than those specified might cause a different freeway lane volume distribution leading to an erroneous calculation.

Sample Problems Using Lane 1 Volume Equations

To understand the use of the equations, several sample problems will be worked:

Sample Problem No. 1A—

Given: The same conditions as in Sample Problem No. 1, the only change being that adjacent upstream and downstream off-ramp conditions are also given, as follows:



The adjacent upstream off-ramp, 1,000 ft away, carries 600 vph; the adjacent downstream off-ramp, 3,000 ft away, carries 400 vph.

Find:

1. The "expected merge" for this on-ramp connection under the given traffic volumes using a lane 1 volume equation.
2. The predicted free-flow merge, using the formula derived from 55 observations.

Solution: The given conditions fall within the requirements for use of Equation 1, so this equation is used to calculate lane 1 volume.

$$\text{Volume in lane 1 (vph)} = -121 + 0.244\,(4,300) - 0.085\,(600) + 640\,\frac{(400)}{(3,000)} = 962$$

Expected merge = 962 vph (lane 1) + 800 vph (ramp) = 1,762 vph

The free-flow merge formula from 55 observations would then be applied as in Sample Problem No. 1 and, because the "expected merge" of 1,762 vph is less than the predicted free-flow merge of 1,782 vph, there should be no congestion. This does not take into account the possible application of the standard errors for the lane 1 volume or the free-flow merge. This aspect is discussed later.

The foregoing answers exhibit little difference from those calculated for Sample Problem No. 1. The most likely reason for the close approximation is that the adjacent ramp conditions used in the equation calculation of lane 1 volume were quite ordinary or average. The difference in the two methods of calculating lane 1 volume becomes more apparent if the adjacent downstream off-ramp carried 700 instead of 400 vph.

Using Equation No. 1, the lane 1 volume would now be 1,026 vph, the "expected merge" 1,826, and the predicted free-flow merge (using the formula from 55 observations) 1,785 vph. The presence of the rather heavy downstream off-ramp, now carrying 700 vph, means more freeway vehicles, in anticipation of exiting, will be using lane 1, thus raising the "expected merge." The "expected merge" now exceeds the free-flow merge so congestion is predicted. The addition of 300 more vph exiting downstream has changed the forecast from free flow to congestion.

If, as before, calculations are also made for 1,000 ramp vph merging into a freeway stream of 4,100 vph and 600 ramp vph merging into a freeway stream of 4,500 vph, using the lane 1 volume formulas which take into account the adjacent ramps, the values are as follows:

Free-Flow Volume (vph)						
			Expected Merge		Predicted Free-Flow Merge	
Total	Freeway	Ramp	400 Vph Exiting Downstream	700 Vph Exiting Downstream	400 Vph Exiting Downstream	700 Vph Exiting Downstream
5,100	4,100	1,000	1,913	1,977	1,815[a]	1,810[a]
5,100	4,300	800	1,762	1,826	1,782	1,775[a]
5,100	4,500	600	1,611	1,675	1,727	1,720

[a]Congestion predicted.

These calculations should be compared with those given earlier.

As more data become available, other equations can be developed to include other adjacent ramp conditions, such as an adjacent on-ramp upstream from the on-ramp under consideration. It is doubtful that equations can be developed which make use of upstream and downstream ramps other than the immediately adjacent ramps. Considerable data are needed to allow accomplishment of such a task. In the interim, if the equations are not applicable to the given freeway layout and volumes, the lane volume curves (Figs. 3, 4, 5, 6, and 7) can be used to determine lane 1 volume just upstream from the ramp nose.

No mention has been made of how to handle the standard error of estimate for a lane 1 volume equation as given in Table 5 (Appendix B). If the objective is to reduce the risk of a failure (i.e., traffic congestion), the logical procedure would be to add the standard error to the calculated lane 1 volume. This would have but a slight effect on the free-flow merge calculation, but it would increase the "expected merge" by the amount of the standard error, thereby decreasing relatively the margin between the "expected merge" and the predicted free-flow merge, for free-flowing conditions.

It should be remembered, of course, that there is also a standard error for the free-flow merge calculation. The two standard errors involved in the two calculations (lane 1 volume and free-flow merge) might be additive in the direction of poorest performance or additive in the direction of optimum performance, or the standard errors might tend to cancel each other out. In the case of operational-type problems, such as ramp closure decisions on congested freeways, it seems sufficient to apply only the standard error of the free-flow merge equation. Subtracting this standard error gives the user a free-flow merge value (a lower limit) which will be exceeded in actual freeway operation approximately 84 percent of the time. In design-type problems, however, some may feel it prudent also to apply the standard error of the lane 1 volume equation. The calculated lane 1 volume increased by the amount of the standard error would then be used in the free-flow merge calculation. The additional measure of safety provided by applying the lane 1 standard error cannot readily be measured in terms of the resulting free-flow merge, but overall the measure of statistical confidence would now be comparable with that usually used in research work—a confidence interval exceeding 90 percent.

Two-Lane On-Ramps

Some ramps designed as 2-lane facilities operate instead as 1-lane, either through lack of demand or because of geometric conditions which make driving them single file more comfortable. Given sufficient demand, operation will become 2-lane. Even 1-lane ramps sometimes operate as 2-lane facilities when high demand forces ramp drivers to double up, as already discussed for the Beaubien ramp in Detroit. For purposes of this report, 2-lane ramps are those designed and operating as such at the terminal of the ramp.

One of the most interesting 2-lane ramps studied was the Northern State Parkway semidirect connection to the Long Island Expressway westbound. This location was studied four times with "peak merge hour" ramp volumes ranging from 2,040 to 2,265 vph. Inasmuch as the Long Island Expressway was opened to a point only 0.6 mi east (upstream) of this ramp at the time of the first two studies, total peak-hour freeway volume for three lanes was only 1,000 to 1,500 vehicles. A few weeks later, five more miles of expressway were opened with peak-hour freeway volume upstream from the ramp of 2,735 vehicles in the morning peak and 1,949 vehicles in the afternoon peak. Operation throughout the peak periods was mostly free-flowing, with only a few slowdowns. Free-flow merging volumes were 2,444, 2,700, 2,468, and 2,688 vph as shown in the following: The 15-min f.f. volumes expanded to 1 hour were as follows:

Study		Free-Flow Volume[a] (vph)							
		Ramp			Freeway				
No.	Time	Lane B	Lane A	Merge[b]	Lane 1	Lane 2	Lane 3	Total	Total[c]
1	A.M.	620	1,376	2,444	448	692	532	1,672	3,668
2	P.M.	1,332	1,160	2,700	208	412	380	1,000	3,492
3	A.M.	684	1,384	2,468	220	1,108	1,076	2,404	4,652
4	P.M.	876	1,528	2,688	284	816	848	1,948	4,352

[a]15-min f.f. volume expanded to 1 hour. [b]Ramp B + Ramp A + Lane 1.
[c]Freeway + Ramp.

The comparatively low freeway volumes permitted ramp lane A traffic to merge directly into lane 2 on occasion. Although the merge volumes given are the additions of the ramp lanes and lane 1, recognition should be given to this role that lane 2 plays when freeway volumes are at low levels. The high merge figures are more understandable when the low-volume nature of the freeway traffic is understood. An acceleration lane 810 ft long was used by the ramp lane B vehicles. In three of the four studies, lane A carried considerably more traffic than lane B. The overall ramp lane volume percentages at ramp volume levels between 1,400 and 2,600 vph are shown in Figure 20 under N.Y.-36a, b, c, d. Evidently, the drivers preferred to take their chances

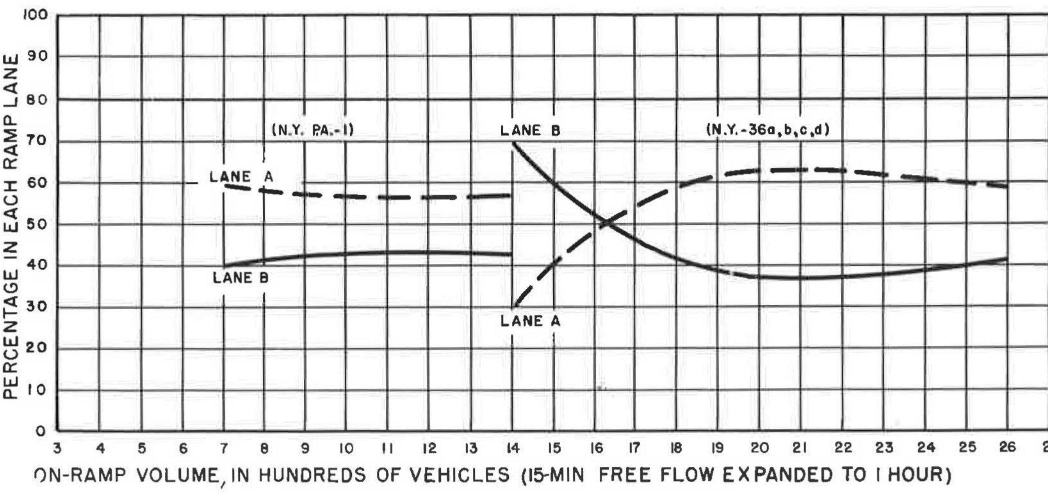


Figure 20. Ramp volume distribution for 2-lane on-ramps (two locations).

in an immediate merge rather than use the acceleration lane and have to find a gap in the already merged lane A and lane 1. The exception was study No. 2, in which 53 percent of the ramp vehicles used lane B. (The difference here might have been caused by a bothersome sun with which the ramp drivers had to contend. At least that is the only reason that can be hypothesized other than the vagaries of traffic.)

All in all, this interchange operated smoothly with going-away free-flow volumes averaging 1,200 to 1,550 vph/lane. It was a case of a ramp of high-volume design dominating the scene, especially in the first two studies. However, since completion of the four studies still another section of the Long Island Expressway has been completed and freeway volumes now are close to practical capacity range as they approach the ramp nose. As might be expected, operation is now congested throughout the peak hour, with backups accumulating on both ramp and freeway.

Another 2-lane ramp of interest was that connecting North Conduit Avenue westbound to Van Wyck Expressway northbound on Long Island. The acceleration lane is 520 ft long. The "peak merge hour" produced the following volumes: Ramp lane B, 1,170; ramp lane A, 1,336; merge (ramp B + ramp A + lane 1), 2,917; lane 1, 411; lane 2, 739; lane 3, 491; freeway (lane 1 + lane 2 + lane 3), 1,641; total (freeway + ramp), 4,147.

Similar to the Northern State Parkway - Long Island Expressway ramp, ramp lane A carried more vehicles than ramp lane B. There were numerous lane changes by ramp vehicles jockeying for position as they approached the nose prior to merging. The merge volume of 2,917 vph seems high, but it should be noted that 2,506 were on the ramp with only 411 in lane 1. Although no data were recorded, a number of ramp vehicles did cut directly into lane 2 of the Van Wyck Expressway, which begins at Idlewild Airport 2.5 mi south of this location.

Because traffic volumes are likely to remain quite stable at this location in contrast to those along the previously discussed Long Island Expressway, free-flowing traffic can probably be maintained with present geometrics. One more note—although accident statistics have not been studied, personal observation of this ramp's operation (and to some extent the Long Island Expressway ramp too) led to the belief that safety is sacrificed when ramp vehicles are given three choices of action: (1) Use of acceleration lane and merge into lane 1; (2) Merge directly into lane 1; and (3) Merge directly into lane 2. The last named action caused most of the near misses observed. Eliminating this type of merge would not be easy, as it is more a result of unusual freeway ramp volume distributions than of geometrics or signing.

A 2-lane diamond ramp studied in New Jersey near the Lincoln Tunnel was the Pleasant Avenue on-ramp to New Jersey Route 3 westbound. The 15-min free-flow volume expanded to 1 hour was: Ramp lane B, 452; ramp lane A, 584; merge (ramp B + ramp A + lane 1), 1,524 (28.6% c.v.); lane 1, 488; lane 2, 1,068; lane 3, 1,028; freeway (lane 1 + lane 2 + lane 3), 2,584; total (ramp + freeway), 3,620 (20.2% c. v.).

Principally because a long upgrade on Route 3 ended only 600 ft upstream from the ramp nose, 80 percent of the commercial vehicles were in lane 1. These slow-moving trucks created a number of large gaps, so that once again ramp vehicles favored lane A (N.Y. P.A. - 1 Study in Fig. 20). Ramp lane B vehicles were hampered by a short acceleration lane (180 ft) and by lane A vehicles that were accelerating after direct entry onto lane 1. Observers commented that as many as 20 percent of the merged ramp vehicles moved over into lane 2 within 200 ft of the nose.

The conclusions that can be drawn from the 2-lane on-ramps studied are as follows:

1. Except when the beginning of the freeway is and will remain a short distance upstream, an extra through lane should be added to the freeway.
2. Downstream (after merge) going-away averages exceeding 1,500 vph/lane will usually result in congestive operation. The most probable reason for this is the difficulty in achieving equitable volume distribution among the freeway lanes downstream from high-volume 2-lane ramps. It follows that any time freeway lane averages approaching 1,000 vph/lane upstream from the ramp are expected, an extra through lane should be added to the freeway at 2-lane on-ramp connections. This is especially so for 4- or 6-lane divided freeways.
3. Addition of an extra through freeway lane would help eliminate direct merging into lane 2 by ramp vehicles and increase overall safety, because only one ramp lane would need to merge into lane 1.

OFF-RAMPS

One of the main objectives of the Freeway Ramp Capacity Study is to determine formulas for computing the capacity of the diverging movement from the freeway to the ramp. There is also a need for determining the relative strength of the roles played by such geometric and traffic characteristics as angle of divergence, sight distance, length and shape of deceleration lane, percentage of commercial vehicles, and lane volume distributions. Although work on these objectives is under way, it has not progressed to the point where results can be reported. The observations presented herein are therefore mostly general impressions developed from a review of the off-ramp data submitted.

The capacity problems found at exit ramps are quite dissimilar from those experienced at on-ramps. Whereas the on-ramp driver has the very real task of choosing a gap and merging into it, no such complicated maneuver is necessary at exit ramps. It is rather disconcerting, therefore, that some off-ramps operate in an unsatisfactory manner.

At off-ramps there are three possible capacity limitations, as follows (circled numbers 4, 5, and 6, Fig. 1):

4. The diverging movement from the freeway to the ramp.
5. The ramp proper.
6. The ramp terminal connection to the street system.

The capacity of the diverging movement from the freeway to the ramp has the greatest effect on the through freeway lanes. Reasons for unsatisfactory diverging maneuvers are sometimes difficult to pin down because the origin of the trouble may be some distance upstream or downstream from the ramp.

Some of the causes of diverging difficulties are as follows:

1. Poor signing and/or sight distance, causing abrupt maneuvers or speed changes close to the exit ramp nose.
2. Lack of adequate weaving length on the freeway upstream from the ramp exit, causing excessive lane changing near the nose. Even though advance overhead signing is present, television surveillance lane-changing studies in Detroit disclosed that the

Figure 21. Crowding in at exit nose, illustrating failure to use auxiliary lane to best advantage.

greatest amount of lane-changing took place just upstream of the off-ramp nose. To a certain extent this prevents maintenance of the uniform speed necessary for smooth operation of exiting traffic.

3. The occasional poor usage of auxiliary and deceleration lanes. Cutting-in at the nose of cloverleaf inner loop off-ramps was a problem at several locations, even though an auxiliary lane was available for use by exiting drivers (Fig. 21).

4. Poor operating characteristics of the ramp proper, causing speed reduction to extend back onto the freeway exit lane.

The concept of a capacity of the ramp proper (No. 5, Fig. 1) is really no different than the capacity of the ramp proper for an on-ramp. It is the physical ability of the ramp to handle a continuous supply of vehicles, assuming that there are no limitations at the ramp terminals. It would seem, therefore, to be dependent on such physical characteristics as radius, width, superelevation, and riding surface. Because this ultimate capacity is so seldom reached in practice, this seems to be an area where controlled laboratory experiments, or perhaps simulation, are needed. Of course, if the ramp is tangent, the ramp proper capacity should be the same as conventional freeway lane capacity for the given speed.

The last mentioned capacity limitation (the ramp's connection to the street, frontage road, or interchanging highway system) is a subject in itself. Although some backup along the ramp can be tolerated, the situation reaches serious proportions when the freeway lane 1 is encroached upon. If the exit ramp's terminal is a merging operation into another freeway or expressway (No. 6, Fig. 1), there are not apt to be serious backups unless the traffic is so heavy on the other facility that the ramp vehicles cannot merge without delay. If the ramp is two lanes wide for storage purposes, backups can usually be confined to the ramp. From an operational standpoint, especially at the divergence from the freeway, a 2-lane ramp may be less desirable than a 1-lane ramp. Diamond ramps are almost always 1-lane. It is usually at diamond interchanges controlled by traffic signals that backups become serious enough to extend back onto the freeway. Occasionally, backups occur at diamond off-ramps controlled by stop signs, usually because of difficulties encountered by left-turning vehicles.

Freeway Volume Distribution at Off-Ramps

Freeway volume distributions at off-ramps for 4-, 6-, and 8-lane freeways are given by lanes in Figures 22, 23, and 24. These distributions were taken just downstream from the ramp nose after the ramp traffic had diverged from the freeway stream.

The curves for 6-lane freeways at off-ramp locations (Fig. 23) are derived from 6-lane freeway volume distributions at all types of off-ramps. This figure also gives the volume distributions where an auxiliary lane is present between the off-ramp and the adjacent upstream on-ramp. The auxiliary lane evidently opens up lane 1, because

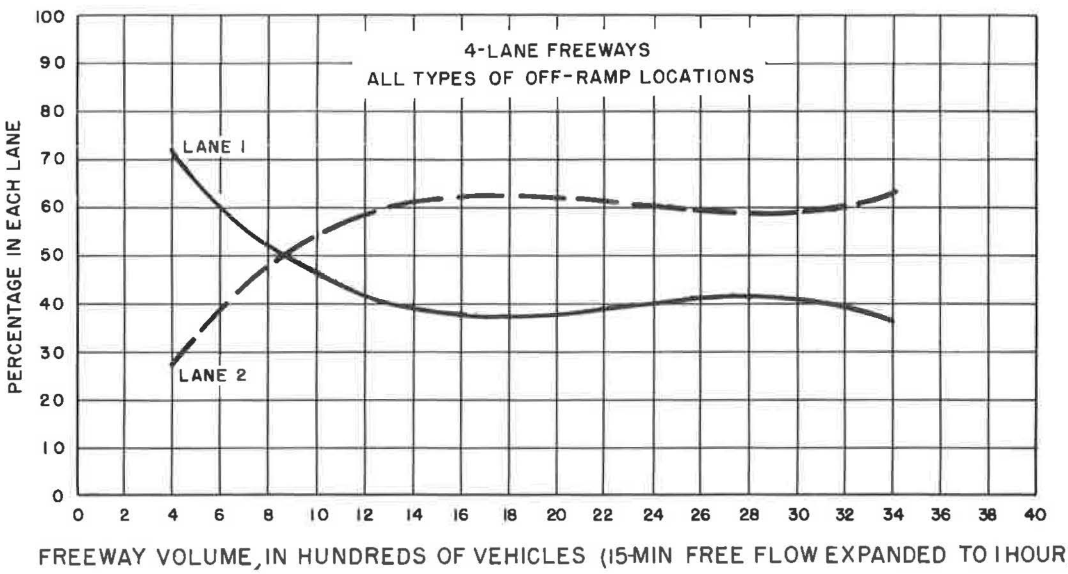


Figure 22. Volume distribution on 4-lane freeways downstream from off-ramps.

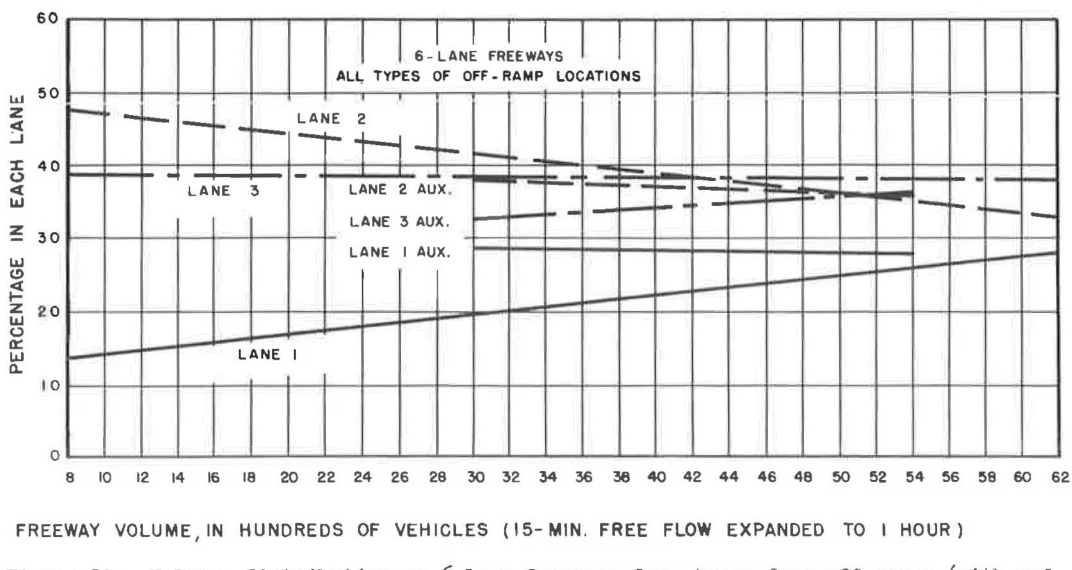


Figure 23. Volume distribution on 6-lane freeways downstream from off-ramps (with and without auxiliary lane upstream).

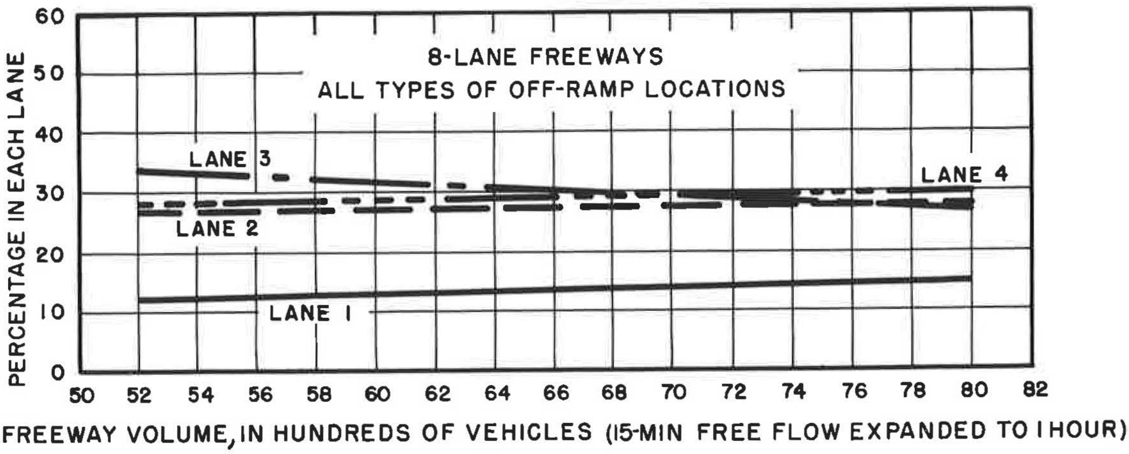


Figure 24. Volume distribution on 8-lane freeways downstream from off-ramps.

where such a lane exists percentages in lane 1 downstream from the off-ramp are 2 to 9 points higher within the volume range shown by the curves. The increased use of lane 1 could also be partially caused by the relative nearness of the adjacent upstream on-ramp.

As stated previously, the distributions shown in Figures 22, 23, and 24 are for the freeway downstream from the ramp nose. This explains the low percentage of freeway traffic in lane 1. If the lane percentages were shown as taken upstream from the ramp (before movement to the deceleration lane or ramp by drivers intending to exit), lane 1 would usually carry the highest percentage of the freeway volume at lane volumes below practical capacity. The bunching of vehicles in lane 1 upstream from exit ramps is not a desirable characteristic. The seeming inability of the freeway traffic to evenly distribute among lanes upstream from off-ramps at volumes below practical capacity is being investigated as a primary contributor to congestion at exit ramps. An exit ramp which requires a considerable reduction in speed at the diverge from lane 1 is especially apt to cause erratic operation.

At volumes above practical capacity, a more ideal utilization of the freeway lanes upstream from exit ramps is accomplished. Lane 1 will often carry the lowest percentage; this is good, because it is the "action lane" subject to the most disturbance from the ramp vehicles.

Vehicle Storage at Off-Ramps

As previously mentioned, one of the problems frequently confronting traffic engineers is that of alleviating major backups on diamond off-ramps. Sometimes the remedy is merely to provide sufficient turning lanes to help in getting the traffic off the ramp and onto the surrounding street system. In other cases, heavy traffic on the surrounding street system complicates the problem so that a solution must be found to absorb the ramp traffic into the local traffic without unduly disrupting overall flow. A good signal system is important, but even at its optimum setting it is not always possible to keep ahead of the high exit volumes encountered over short periods. There must be room to store these vehicles in the interim.

Tying a diamond ramp to a parallel frontage road, either continuous or non-continous, appears advantageous when the frontage road is not heavily used by through vehicles. In such cases, where adequate weaving distance is available between the ramp-frontage road junction and the cross street, maneuvering of the ramp vehicles will be facilitated and serious backups will be eliminated. On the other hand, the connection of diamond ramps to heavily used frontage roads at points only a few hundred feet from the cross street was a cause of trouble at several study locations; not only did the frontage road vehicles monopolize the green signal time, but they hindered ramp vehicles attempting to obtain access to the desired frontage road lane. Ramp drivers wishing to turn right at the cross street were especially hampered.

The operational problems and capacity limitations at diamond interchanges are important enough to warrant research projects in their own right. Highway Research Board Bull. 291 contains a recent research study (4) along these lines.

ACKNOWLEDGMENT

The nomographs presented in this text were derived by Mrs. Beverly Norris, Highway Research Engineer, Traffic Research Branch, and Steiner Silence, Traffic Engineer, Region 4, U.S. Bureau of Public Roads.

REFERENCES

1. Amer. Assoc. of State Highway Officials, "A Policy on Geometric Design of Rural Highways." (1954).
2. Amer. Assoc. of State Highway Officials, " A Policy on Arterial Highways in Urban Areas." (1957).
3. Newman, L., "Traffic Operation at Two Interchanges in California." HRB Record 27 (1963).
4. Capelle, D.G., and Pinnell, C., "Capacity Study of Signalized Diamond Interchanges." HRB Bull. 291, pp. 1-25 (1961).

Appendix A

REGRESSION ANALYSIS

Discussion of Variables

The problem to be solved is the determination of free-flow merge volumes for various freeway and ramp volumes and for various geometric configurations. A regression analysis for one-lane right-hand on-ramps was run using 73 observations (Table 4) from 50 locations. Lack of sufficient two-lane on-ramp studies prevented any reliable regression analysis in that category.

All types of right-hand on-ramps were included in the one-lane on-ramp regression analysis. These included 16 diamonds, 12 cloverleaf outer connections, 6 cloverleaf inner loops, 8 slips, 3 directs, 4 semidirects, and 1 trumpet outer connection. A number of these ramps were studied several times, not only during morning and evening rush hours but also on different days. As discussed later, several of these ramp types were later deleted, finally reducing to 55 observations, from which an additional more limited formula was obtained.

The formulas contain six independent variables expressing traffic characteristics, geometrics, and community size. The glossary should be consulted for detailed definitions of these variables. The dependent variable, free-flow merge volume in vph, is the unknown quantity desired. It should be emphasized that computations are made for merge capacity and not primarily for the number of ramp vehicles which can enter onto the freeway without congestion. Once the free-flow merge has been computed, and it should be kept clearly in mind that this is not a constant for all combinations of traffic, the only remaining step necessary is to subtract the lane 1 volume from the merge volume to determine the ramp's capacity at the given freeway volume.

Some explanation of the independent variables is in order. One might wonder why X_1, the percent freeway utilization, is used as a variable. This is necessary as a reflection of demand. As a "plus" quantity, the higher the freeway utilization, the higher the free-flow merge volume determined by the formula. At very high freeway volumes, say 90 percent freeway utilization, a high merge volume can be expected, although most of these merging vehicles will be in lane 1. The volume which could be accommodated by the ramp would be low in this case.

TABLE 4

FREE-FLOW MERGE REGRESSION ANALYSIS

Item	Variables						
	Y F.F. Merge	X_1 % Fwy. Util.	X_2 % Comm. Veh. in Merge	X_3 Ramp/Merge Ratio	X_4 Angle of Convergence	X_5 Length of Accel. Lane	X_6 Metrop. Area Pop.[a]
73 Observations[b] (Eq. A):							
Type of variable	Dep.	Indep.	Indep.	Indep.	Indep.	Indep.	Indep.
Units	Vph[c]	% x 100	% x 100	% x 100	Degrees	Feet	1,000's
Mean	1569.2	67.9	4.8	38.0	13.9	418.2	2549.7
Std. deviation	287.6	14.9	4.2	15.4	10.4	258.5	1462.2
Net regression coefficient	1.0	+8.5	-16.5	+7.6	-1.0	+0.22	+0.071
Std. error of net regression coefficient	0.0	2.2	5.4	1.9	2.2	0.09	0.019
Partial determination coeff.	1.0	0.19	0.13	0.20	0.00	0.08	0.17
Level of significance	–	0.01	0.01	0.01	–[d]	0.02	0.01
55 Observations[e] (Eq. B):							
Mean	1616.3	69.9	4.8	38.7	12.1	419.5	2768.2
Std. deviation	277.3	15.4	4.5	14.9	9.5	267.8	1451.0
Net regression coefficient	1.0	+10.1	-17.9	+9.5	-4.8	+0.14	+0.068
Std. error of net regression coefficient	0.	2.3	5.7	2.3	2.6	0.10	0.020
Partial determination coeff.	1.0	0.28	0.17	0.26	0.07	0.04	0.19
Level of significance	–	0.01	0.01	0.01	0.10	0.20	0.01

[a]Value used in formula should not exceed 5,000. [b]Const. = 527.7, R^2 = 0.68, std. error of Y = 169.8, std. error/mean = 0.108. [c]15-min f.f. expanded. [d]Not significant. [e]Const. = 441.3, R^2 = 0.71, std. error of Y = 158.9, std. error/mean = 0.098.

X_2, the percent commercial vehicles in the merge (% c.v. in merge), is a "minus" quantity. The simple negative correlation between this variable and the free-flow merge is shown in Figure 25 with cloverleaf inner loops treated as a separate category from other types of ramps.

X_3, the ramp volume/merge volume ratio, is another traffic characteristic of primary importance because it is an indication of ramp demand and availability of gaps in lane 1. This ratio has already been discussed in connection with the proportions of ramp and lane 1 volumes for free-flow merge, as shown in Figures 13 and 14.

The geometric variables used are X_4, angle of convergence, and X_5, length of acceleration lane. The former has a minus correlation, as shown in Figure 26. For X_5, the free-flow merge volume has a positive correlation with the length of acceleration lane (Fig. 27). Although the regression analysis is concerned exclusively with one-lane on-ramps, several 2-lane ramps were included in the data determining the curve in Figure 27. Accordingly, the maximum free-flow merge of 2,100 vph is not outside capability, because the 2-lane ramps had long acceleration lanes. Also, some ramp

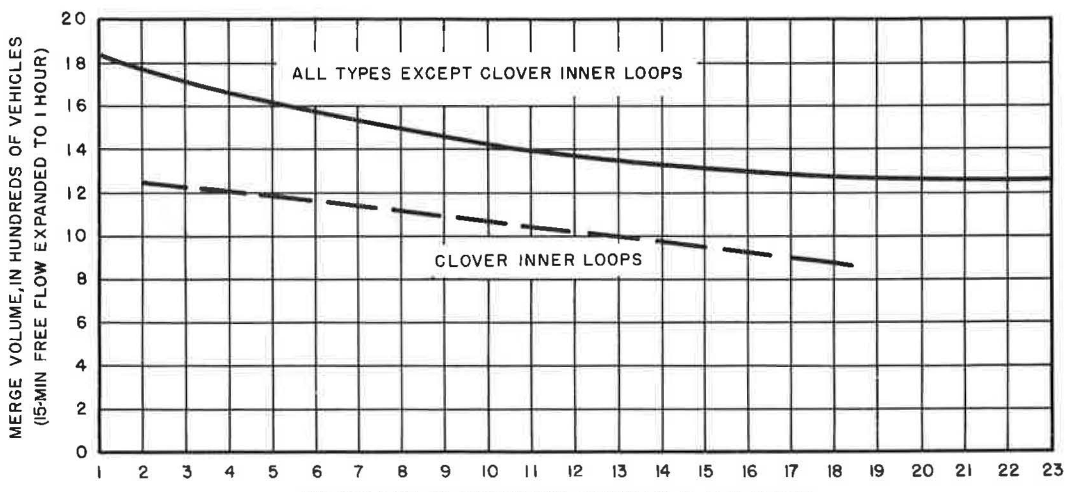


Figure 25. Free-flow merge volumes vs percent commercial vehicles in merge.

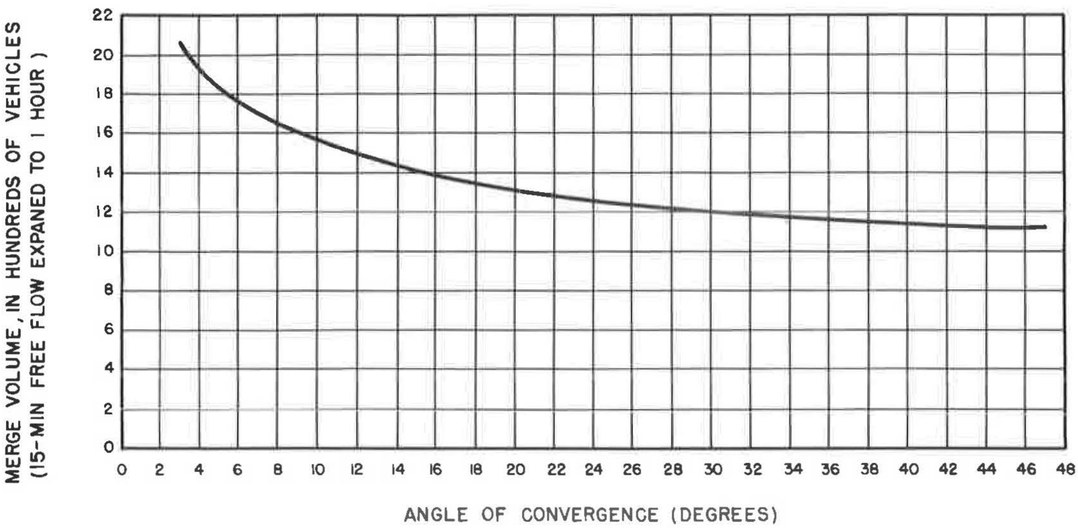


Figure 26. Free-flow merge vs angle of convergence of on-ramp with freeway.

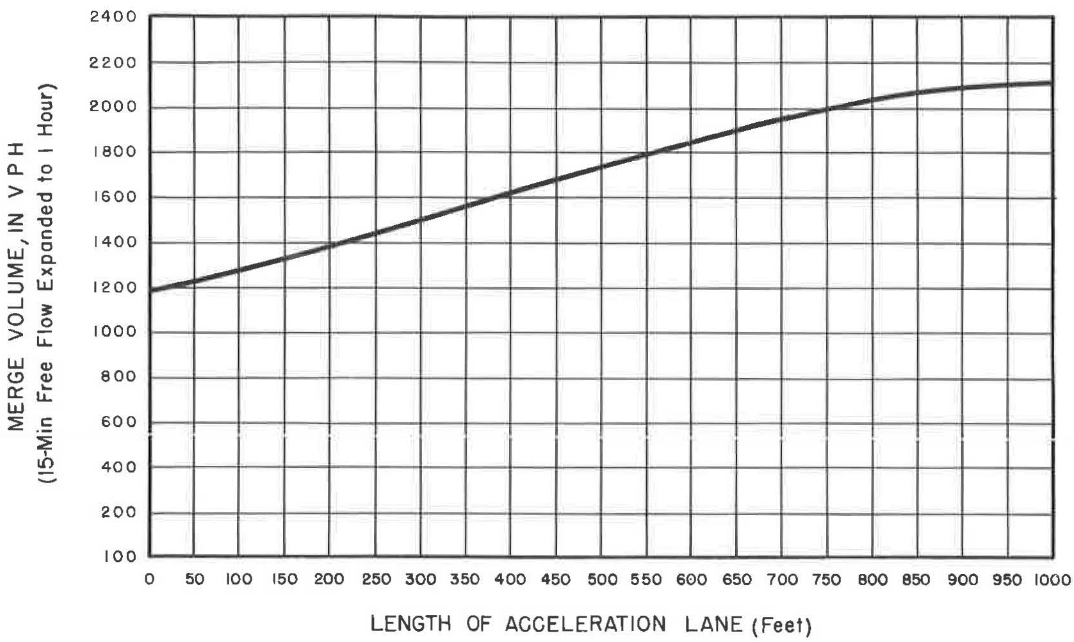


Figure 27. Free-flow merge volume related to length of acceleration lane for 1-lane and 2-lane on-ramps, combined.

vehicles actually merged directly into lane 2, because the freeway volumes were in several cases quite low.

These two geometric variables, X_4, angle of convergence, and X_5, length of acceleration lane, admittedly do not cover all the geometric features of ramp-freeway connections. To name a few, consideration could be made of the width of ramp, shoulders, grades, and particularly sight distances. None of these had much variation in the 73 observations. The effect of a steep sustained uphill grade can be substantial where commercial vehicles are involved, but the studies submitted for this project contained few freeway grades exceeding 2 percent. Grades were considered in the analysis, but no significant relationship was found, although it is possible that this could have been a significant variable if there had been more variation in grades among the study locations. An upgrade ramp with poor sight distance certainly does not lend itself to free-flow operation, but these are not common on modern freeways and none were included in the study.

Sight distance at volume levels below practical capacity is undoubtedly important even when the ramp driver is still back 400 ft from the ramp nose. However, when volumes increase to the levels usually found during peak hours, the on-ramp driver faces a later decision in sizing up lane 1 traffic, now heavier, but traveling at a more uniform and usually slower speed than at volumes below practical capacity. The driver necessarily has to limit his vision to those lane 1 vehicles which are within several hundred feet of the ramp nose. At higher ramp volumes, the ramp driver must also be more concerned with fellow ramp drivers, especially those immediately ahead of him. In essence, sight distance for the last 200 ft traversed along the ramp is all that really matters at high-volume levels. Sight distances at practically all the study sites were adequate in this sense—the driver could see as much as he needed to see as he moved along the ramp just before entering the acceleration lane or lane 1 (if an acceleration lane was unavailable). Sight distance, as such, is really accounted for in the two geometric variables. A narrow angle of convergence usually helps simplify the task of sizing up gaps in lane 1, whereas the presence of an acceleration lane assures the driver of the necessary time to see and choose a gap for merging.

The use of X_6, metropolitan area population expressed in thousands, might be questioned because it is outside the scope of geometrics and traffic characteristics usually associated with capacity calculations. Past experience in intersection capacity pointed toward its inclusion and the results of the analysis, where it was given a trial deletion, confirmed the need for this variable.

Some other possible variables which were tried and found insignificant, within the limits of the study, were the number of freeway lanes in one direction and the percentage of commercial vehicles in the freeway lanes other than lane 1.

Effect of Variables

Table 4 presents a more detailed look at the variables used and their relative significance. The R^2, the coefficient of determination or measure of explained multiple correlation for the entire formula, of 0.68 was slightly exceeded several times in the computer runs for 73 observations. However, the formula chosen was the best from the standpoint of practical application and its standard error of estimate of 169.8 vph was only a few vehicles per hour higher than the lowest standard error of estimate obtained from the various runs. The partial determination coefficients (r^2) for each of the variables are also given.

Using the student's "t" test, all of the variables except X_4, angle of convergence, were found significant at the 0.02 level. It was decided to retain X_4 because, from practical experience, it seemed that this variable should be important. As is discussed later, it is significant at the 0.10 level after deletion of 18 observations comprising the cloverleaf inner loops and slip ramps. Accordingly, it is retained as a variable in each of the formulas.

One of the most significant variables is X_6, the metropolitan area population. Undoubtedly, a major reason for this importance is less fluctuation in demand in the larger cities, enabling a more stable flow at given volume levels. Figure 2, which shows the percent of the peak hour of the peak 5-min period within the peak hour, indicates less pronounced short-period peaking in the large cities.

Too, there are other factors aside from less fluctuation in demand in the large cities. One of the these is driver experience, a necessary part of smooth high-volume operation. Drivers in the large metropolitan areas have had much more freeway experience in the past decade than their counterparts in the smaller cities such as Jacksonville, Fla., or Columbus, Ohio. On a daily driving basis, the larger cities with their more extensive and often better designed freeway networks provide more merging situations. In a city like Chicago, the driver is expected to merge into the traffic stream without stopping, and he does. In a smaller city this compulsion is not so great. A stopped car often blocks those following and the low entry speed of the resultant queue tends to cause congestive operation at lower volume levels. Time and again, observers in the smaller cities commented on the unnecessary stops by ramp vehicles. As the Interstate urban sections are completed, driver skills should develop, but even after the completion of the Interstate system, the large cities should still offer more daily freeway driving and a more uniform flow of traffic at given volume levels. Aside from experience, but probably closely allied to it, is the aggressiveness which characterizes big city drivers. This aggressiveness is part of the generally faster pace of life in the large cities and is certainly not a deficiency in courtesy. In fact, there is probably more beneficial give-and-take driving in the larger cities.

However, in using the formulas, a metropolitan area population exceeding 5,000,000 would add an inordinate amount of vehicles to the answer. This would indicate more "merging ability" superiority than actually exists for the few cities with very large metropolitan area populations, such as New York. For this reason, the largest X_6 value used should not exceed 5,000. This figure should be used for New York City, Los Angeles, and Chicago, the only metropolitan areas exceeding 5,000,000 population.

The designer might be in a quandary as to the population figure which should be assigned to an interchange located outside city limits; for example, one located in an unincorporated area of New Jersey near New York City. In such a case, judgment would have to be used in determining the type, the origin, and the destination of drivers using the interchange. For instance, if the interchange was near Ridgefield Park, primarily serving commuters of Passaic and Bergen counties enroute to and from Hudson County, N.J., and New York County (Manhattan), the designer could use the combined populations of these counties. For Bergen, Passaic, Hudson, and New York Counties, this total would be 3,495,888 from the 1960 census. Accordingly, 3,496 would be entered as the X_7 value.

Deletion of Observations

To obtain a more homogeneous sample, the ramps of very short length and the ramps of sharp curvature or short radius near the nose were deleted from the 73 observations. The ramps having these features were the slip and cloverleaf inner loop ramps. Deletion of these ramps left a remainder of 55 observations comprised of 27 diamonds, 17 cloverleaf outer connections, 6 semidirects, 4 directs, and 1 trumpet outer connection. These observations were from 37 locations.

The formula obtained from the 55 observations had an R^2 of 0.71 and a standard error of estimate of 158.9 vph. The mean input value of the free-flow merge was 1,616 vph, or 47 vph more than the mean for the 73 observations. Because the deleted slip-ramps and cloverleaf inner loop ramps were generally of poorer geometrics, this difference is as expected.

The student's "t" tests for significance of variables disclosed significance at the 0.01 level for all the variables except X_4 and X_5. These two variables, the angle of convergence and the length of acceleration lane, were significant at the 0.10 and 0.20 levels, respectively. X_4, the angle of convergence, now has more importance than formerly with a coefficient of -5.0. For the 55 observations, the average angle of convergence was 8° for diamond ramps and 22° for cloverleaf outer connections. If only the angle of convergence is considered, there would be an average difference, attributable to the X_4 coefficient of -5.0, of 70 vph in the free-flow merge volume for the diamond and cloverleaf outer connection ramps included in the analysis. The foregoing is stated only to give a general indication of the contribution of X_4 to the formula. As in any multiple regression formula, final conclusions should not be based on an interpretation of the effect of any single variable. Primary importance should be attached to the computed free-flow merge which takes into account all the variables in the formula.

Appendix B

LANE 1 VOLUME EQUATIONS

In freeway driving, as well as in any other type of driving, drivers strive to obtain a certain degree of comfort or freedom. The hoped-for optimization of this freedom is attempted by the individual driver by choosing the traffic lane which promises the least conflict compatible with his intended destination. The reactions of each driver and the distribution of the freeway traffic in general are primarily dependent on the freeway volume level. It is unlikely that a six-lane freeway would carry 1,900 vph in lane 3 while carrying only 1,000 vph in lane 2, although such a condition is entirely possible without a breakdown in either lane. Most certainly, some of the drivers in lane 3 would move to lane 2 to achieve more comfortable headways. In the same sense, lane 1 volume is also primarily dependent on the freeway volume; this is especially the case just upstream from an on-ramp connection. Downstream from the ramp, the volume in lane 1 would, to a considerable extent, be dependent on the newly merged ramp volume.

The use of curves for lane volume percentages, based on varying freeway volumes, is one method of determining the lane 1 volume used in the free-flow merge calculations. Although the curves are based on freeway volumes, they represent averages of all the other factors which influence the use of lane 1. Some of these factors could be signing, adjacent ramp volume and distance, study ramp pressure on lane 1, commercial vehicles (especially where sustained upgrades are encountered), locality of the interchange, trip lengths, and the spacing of interchanges. These determinants, although not as important as the total freeway volume, nevertheless have some effect on the freeway volume distribution.

Accordingly, another method, applicable within the limits of the available data, is the use of lane 1 volume equations developed by multiple regression analysis. These equations contain not only the freeway volume as a variable, but also the ramp volume and adjacent ramp action. Use of these additional factors makes possible an increase in the accuracy of lane 1 volume calculations, especially at freeway volume levels below practical capacity. Nomographs (Figs. 10, 11, and 12) representing Eqs. No. 1, 4, and 5 are available for graphic solution of problems.

The available data were sufficient for only the more common freeway layouts, as shown by the sketches in Table 5 and the adjacent ramp distance and volume ranges for each equation as given in Table 3.

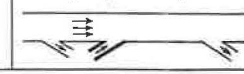
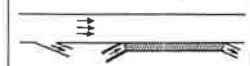
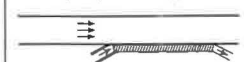
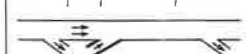
Equation No. 1

Eq. No. 1 (Table 5) is used for determining the lane 1 volume upstream from the on-ramp nose for 6-lane freeways where there are adjacent upstream and downstream off-ramps. There is no auxiliary lane between the on-ramp and the adjacent downstream off-ramp.

The data used to develop this equation consisted of 325 free-moving 5-min traffic counts from the eastern end of the Edsel Ford Expressway in Detroit, from the Gulf Freeway in Houston, and from the Cross Island Parkway on Long Island. The Detroit data consisted of 266 5-min traffic counts from nine on-ramp locations and their adjacent ramps which were counted simultaneously. These on-ramps consisted of 6 diamonds, 1 direct connection, and 2 cloverleaf outer connections. The cloverleaf outer connections were from a partial cloverleaf interchange and, although there were adjacent upstream inner loop off-ramps, there were no upstream inner loop on-ramps to cause weaving at the interchange. The 6-mile freeway section in Detroit was a smoothly operating freeway at high volumes and quite typical of a modern radial depressed facility. It was opened to traffic in 1958-9. The Houston ramps were both slip types, which provided 42 5-min traffic counts. The Long Island ramp was a cloverleaf outer connection with no upstream inner loop on-ramp. The 5-min counts used in the regression analysis were from periods when the traffic was moving steadily without stop-and-go operation. Naturally enough, at volumes near possible capacity the speeds could be in the 25- to 30-mph range. It was decided to include some vol-

TABLE 5

FORMULAS FOR LANE 1 VOLUME

6-LANE FREEWAYS	VARIABLES								Const	R^2	Std. Error of Y v.p.h.	Std. Error Mean
	DEPENDENT	INDEPENDENT										
	Y Volume in Lane 1 v.p.h.	X_1 Freeway Volume v.p.h.	X_2 Ramp Volume v.p.h.	X_3 Distance in Feet to Adjacent Upstream Off-Ramp	X_4 Vol. of Adjacent Upstream Off-Ramp v.p.h.	X_5 Distance in Feet to Adjacent Downstream Off-Ramp	X_6 Vol. of Adjacent Downstream Off-Ramp (v.p.h.)	$\frac{X_6}{X_5}$				
With Adjacent Off-Ramps									-121	.80	140	.134
Mean	1041	4327	594	1459	465	2482	449	--				
Standard Deviation	312	943	349	558	251	1404	260	--				
Range of Use of Variable in Equation		2400 - 6200	100 - 1700	900 - 2600	50 - 1100	900 - 5700	50 - 1300	--				
Net Regression Coefficient		+.244			-.085			+640				
Std. Error of Net Regr. Coefficient		.008			.035			50				
Partial Determination Coefficient		.72			.02			.34				
Level of Significance		.01			.01			.01				
Equation No. 1	Vol. in Lane 1 (v.p.h.) = -121 + .244 (fwy. vol.) - .085 (vol. of upst. off-ramp) + 640 (Vol. of Downstream off-ramp) / (Dist. to Downstream off-ramp)											
With Adjacent Upstream Off-Ramp and Auxiliary Lane Downstream									+62	.77	139	.127
Mean	1099	3876	638	1513	407	775*	496	--				
Standard Deviation	287	1050	363	703	232	160	187	--				
Range of Use of Variable in Equation		1900 - 6200	150 - 1900	450 - 2150	50 - 1000	550 - 950	50 - 1000	--				
Net Regression Coefficient		+.232	-.072	-	-	-.041	+.432	--				
Std. Error of Net Regr. Coefficient		.015	.047	-	-	.108	.068	--				
Partial Determination Coefficient		.67	.02	-	-	.00	.25	--				
Level of Significance		.01	.01	-	-	N.S.**	.01	--				
Equation No. 2	Vol. in Lane 1 (v.p.h.) = 62 + .232 (fwy. vol.) - .072 (ramp vol.) - .041 (length aux. lane) + .432 (vol. of downstream off-ramp)											
Auxiliary Lane Downstream No Significant Upstream Adjacent Ramp									-160	.80	174	.154
Mean	1130	4062	678	Not	Net	756*	492	--				
Standard Deviation	387	1138	363			149	205	--				
Range of Use of Variable in Equation		1900 - 6200	50 - 1900			550 - 950	50 - 1000	--				
Net Regression Coefficient		+.273	-.195	Considered	Considered	-	+.633	--				
Std. Error of Net Regr. Coefficient		.014	.042			-	.070	--				
Partial Determination Coefficient		.71	.12			-	.35	--				
Level of Significance		.01	.01			-	.01	--				
Equation No. 3	Vol. in Lane 1 (v.p.h.) = -160 + .273 (fwy. vol.) - .195 (ramp vol.) + .635 (vol. of downstream off-ramp)											
Cloverleaf Inner Loop with Auxiliary Lane to Inner Loop Off-Ramp									-87	.64	178	.203
Mean	878	3792	810	Not	Not	-	447	--				
Standard Deviation	293	790	383			-	282	--				
Range of Use of Variable in Equation		2?00 - 5600	200 - 1500			450 - 850	150 - 1500	--				
Net Regression Coefficient		+.225	-.140	Considered	Considered	-	+.500	--				
Std. Error of Net Regr. Coefficient		.019	.040			-	.054	--				
Partial Determination Coefficient		.51	.08			-	.39	--				
Level of Significance		.01	.01			-	.01	--				
Equation No. 4	Vol. in Lane 1 (v.p.h.) = -87 + .225 (fwy. vol.) - .140 (ramp vol.) + .500 (vol. of downstream off-ramp)											
4-LANE FREEWAYS												
With Adjacent Off-Ramps									+55	.92	76	.086
Mean	874	2304	436	716	166	2532	259	--				
Standard Deviation	270	672	239	462	140	1142	167	--				
Range of Use of Variable in Equation		1100 - 3700	50 - 1000	100 - 1450	50 - 600	1000 - 5000	50 - 750	--				
Net Regression Coefficient		+.363	-.184	-	-	+.022	+.030	--				
Std. Error of Net Regr. Coefficient		.009	.028	-	-	.007	.044	--				
Partial Determination Coefficient		.90	.20	-	-	.05	.00	--				
Level of Significance		.01	.01	-	-	.01	N.S.**	--				
Equation No. 5	Vol. in Lane 1 (v.p.h.) = 55 + .363 (fwy. vol.) - .184 (ramp vol.) + .022 (dist. to downstream off-ramp) + .030 (vol. of downstream off-ramp)											

*Same as length of auxiliary lane.
**Not Significant.

umes as high as possible capacity in the analysis as long as the freeway traffic was moving in a steady manner without stop-and-go.

Except for the combination of the downstream off-ramp volume and distance in the last term, each variable was assumed linear in the multiple regression analysis. Although correlation of the lane 1 volume with the total freeway volume is slightly curvilinear, the relationship is not pronounced enough to warrant transformation of the variable. The R^2 (coefficient of determination or explained variance) of 0.80 and the standard error of 140 vph are indications of a good fit.

Several variables other than those contained in the formula were tried. One was the percentage of commercial vehicles in the merge, which was found insignificant within the limits of the study data. However, commercial vehicles can dominate lane 1 use on sustained upgrades. The angle of convergence of the ramp with the freeway and the length of acceleration lane were also tried as variables, but it was decided that there were insufficient locations to give a valid range of input values. The other variables (not used) which affect lane 1 volume are evidently not too important individually. Some of them, such as trip length, would be difficult to measure or to apply in a formula.

A final word of caution—Eq. No. 1 should not be used for cloverleaf inner loop on-ramps where there is an adjacent downstream cloverleaf inner loop off-ramp. The close-in weaving between the inner loop ramps could change the freeway volume distribution so as to invalidate the equation. As shown in Table 3, the adjacent downstream off-ramp should be a minimum of 900 ft away. This limitation automatically rules out most conventional cloverleaf inner loop connections. Eq. No. 4 can be used at cloverleaf inner loops having auxiliary lanes.

Equation No. 2

As shown in Table 5, Eq. No. 2 is used for determining lane 1 volume upstream from the on-ramp nose where there is an adjacent upstream off-ramp and a downstream off-ramp less than 1,000 ft away connected to the study ramp by an auxiliary lane.

The data used consisted of 128 observations, of which 63 were 5-min periods and 65 were 1-min periods. The 63 5-min periods were from two locations—a cloverleaf inner loop connection, studied both morning and evening peak hours, on the Valley Highway in Denver, and a slip ramp connection on the Gulf Freeway in Houston. The 65 1-min periods were from a diamond connection on the Edsel Ford Expressway in Detroit.

Table 5 indicates that the distance away and volume of the adjacent upstream off-ramp are not contained in the equation. However, as explained in the main text, this upstream off-ramp should be present and within the limits set up in Table 3 before Eq. No. 2 becomes valid. The length of the auxiliary lane, X_5, although not found significant is nevertheless contained in the equation. This logically could be a significant variable but there was not enough variation in the auxiliary lane lengths for the study locations used to prove it so here.

It is rather interesting to note the strong effect (high coefficient) of the downstream off-ramp volume as contrasted with the same variable in Eq. No. 1. The increased effect is apparently because the prospective off-ramp traffic loads up lane 1 just upstream from the on-ramp in anticipation of moving over into the rather short (550 to 950 ft) auxiliary lane. The main on-ramp connection now exerts less pressure on the lane 1 volume than formerly.

Equation No. 3

This equation is used for determining lane 1 volume upstream from the on-ramp nose for 6-lane freeways when the on-ramp under consideration is connected to an adjacent off-ramp less than 1,000 ft downstream by an auxiliary lane. Also, there is either no nearby upstream ramp, or if so its volume or effect on through traffic is negligible.

This equation extends the data contained in Eq. No. 2 to include 35 additional 5-min periods, making a new total of 163 observations. The 35 additional observations come from a cloverleaf inner loop on Long Island and a diamond ramp in Los Angeles. In the case of the Long Island ramp there is no adjacent upstream outer connection off-ramp.

Again, the equation is restricted to locations where the auxiliary lane to the downstream off-ramp is less than 1,000 ft in length.

Equation No. 4

Eq. No. 4, intended for use in determining lane 1 volume on 6-lane freeways just upstream from inner loop on-ramps at cloverleaf interchanges, includes as independent variables the freeway volume upstream from the ramp nose, the inner loop on-ramp volume, and the downstream off-ramp volume. The inner loops should be connected by an auxiliary lane in the usual range of length (450 to 850 ft). The basic data come from loops without outer connections and from loops with outer connections (conventional cloverleafs). Because of lack of volume data, the outer connection operation, when present, was not included as variables in the equation. Undoubtedly, an upstream outer connection off-ramp, if present, would tend to reduce the lane 1 volume at the inner loop nose. This omission, together with the more pronounced natural variation of operation at cloverleafs, helps to account for the higher standard error of estimate at cloverleafs, helps to account for the higher standard error of estimate (178 vph) and lower R^2 (0.64) of this equation as compared with the other equations. This is not to distract from its superiority over using curves based on freeway volume only. The ramp volumes on inner loops have a great effect on the lane 1 volume.

The basic data, consisting of 136 5-min observations, comes from seven locations on Long Island, along the Edens Expressway in Chicago, and from the Whipple Avenue interchange of the Bayshore Freeway near San Francisco.

Equation No. 5

This equation is used for determining lane 1 volume upstream from an on-ramp nose for 4-lane freeways where there are adjacent upstream and downstream off-ramps. There is no auxiliary lane between the on-ramp and the adjacent downstream off-ramp. Thus, the situation is the same as for Eq. No. 1 except that the freeway is 4 lanes instead of 6 lanes.

The data used consisted of 187 5-min traffic counts from seven locations in Denver, St. Louis, San Antonio, and San Jose. These ramps included 4 cloverleaf outer connections, 1 diamond, 1 semidirect connection, and 1 partial cloverleaf inner loop on-ramp with no following inner loop off-ramp. The R^2 of 0.92 and standard error of 76 vph of the equation are excellent, especially considering that only four independent variables are used.

Although the distance to and volume of the adjacent upstream off-ramp are not contained in the equation, such a ramp must be present within the ranges shown in Table 3 before the equation may be applied. Herein lies the biggest weakness of this equation, because the adjacent upstream off-ramp volume could possibly exceed the 50- to 600 vph range specified in Table 3. Volumes and distances outside the specified ranges could have an effect on lane 1 volumes which the equation could not accurately fit. Figure 3 is applicable to situations which fall outside the specified ranges.

Figure 3 should also be used whenever the connection being considered is a cloverleaf inner loop on-ramp, because Eq. No. 4 does not apply to this type of layout. The weaving to the downstream off-ramp causes a different lane 1 volume curve, as shown in Figure 3. However, if the ramp is a cloverleaf outer connection or a cloverleaf inner loop (at a partial cloverleaf interchange) with no adjacent downstream inner loop off-ramp, the equation is applicable.

Appendix C

OPERATION OF A DIAMOND ON-RAMP

Most of the interchanges on the Edsel Ford Expressway in Detroit are of the diamond type. Usually the ramps connect to a service road 300 to 600 ft from the cross street. The eastern end of the Expressway, which was opened to traffic in 1958-9, has some very efficient ramp-freeway connections. The one presented here is the Mt. Elliot on-ramp to the Expressway westbound (inbound in the morning). The study period was from 6:00 to 8:10 a.m. The first hour of operation was free-flowing at speeds of 45 to 55 mph. The interval from 7:03 to 7:09 was a period of gradually decreasing speeds as the freeway became saturated at the merging area. From 7:09 to the end of the study at 8:10, speeds were erratic and there was some stop-and-go traffic. Only a few times, for short intervals only, did speeds get up to 30 mph.

The period from 6:30 to 7:00 was a period of good demand and high volumes at speeds of 45 to 55 mph. Most of the elements of optimum operation are present at this location—level, tangent expressway, narrow angle of convergence (6°), and good sight distance. The ramp had a 500-ft tapered acceleration lane, which is close to the average length of acceleration lane for the various studies submitted nationally. The half-hour period expanded to one hour had the following volumes: ramp, 478 vph, 9.6 percent trucks; lane 1, 1,348 vph, 6.4 percent trucks; merge, 1,826 vph (ramp + lane 1); lane 2, 1,986 vph; lane 3, 2,150 vph; freeway, 5,484 vph (lane 1 + lane 2 + lane 3); total, 5,962 vph, 2.3 percent trucks (freeway + ramp); going-away average per lane = 1,987 vph.

During the half-hour period, 378 vehicles entered the service road at the cross street with 239 of these going down the ramp and 139 staying on the service road past the ramp. In other words, approximately 63 percent of the vehicles on the service road used the ramp during free-flow freeway operation.

Nearby ramps counted at the same time were the upstream off-ramp (1,580 ft away) to Mt. Elliot with an expanded volume of 320 vph and the downstream off-ramp (1,860 ft away) to Chene with an expanded volume of 598 vph. It might be noted that, using Eq. No. 1 the expanded volume calculated for lane 1 would have been 1,396 vph, which is 48 vph more than the 1,348 vph actual count. This is a 3.6 percent error, which in this case would cause a slightly higher computed "expected merge" volume. Weaving was quite noticeable downstream from the study ramp location as vehicles in lanes 2 and 3 moved over to exit at the Chene ramp.

At the main study ramp the flow rate (total freeway stream after merge) for each of the six 5-min periods in the half hour considered was 5,604, 5,340, 5,736, 5,868, 6,276, and 6,984 vph. As can be seen, the traffic buildup was steady to the breakdown point, which occurred 3 min after the 5-min period during which the 6,984-vph rate was recorded. Because the breakdown definitely occurred at the study location, the 2,316-vph/lane average for the last 5-min period may have some significance as representing a maximum 5-min capability downstream from a ramp. Short-period volumes in this range have been obtained at other smoothly operating sections.

As mentioned before, the period from 7:10 to 8:10 was a period of erratic and low-speed traffic flow. The following volumes for this hour illustrate quite well that as long as demand is steady and continuous heavy volumes of traffic can pass a point even under these conditions: ramp, 842 vph, 8.8 percent trucks; lane 1, 1,378 vph, 6.0 percent trucks; merge, 2,220 vph (ramp + lane 1); lane 2, 1,993 vph; lane 3, 2,008 vph; freeway, 5,379 vph (lane 1 + lane 2 + lane 3); total, 6,221 vph, 2.6 percent trucks (freeway + ramp); going-away average per lane, 2,074 vph.

Of the 1,743 vehicles on the service road during this hour period, 842 went down the ramp. This 48 percent use of the ramp, as contrasted with 63 percent use during free flow, suggests that some drivers stayed off the expressway because of its congested state. The option afforded the service road driver is one of the big advantages of connecting diamond ramps to service roads rather than directly to the cross-street.

Finally, mention should be made of some high lane counts per 5-min period. During the free-flow period, lane 2 had 5-min counts of 182 and 198 vehicles and lane 3

Figure 28. Overhead signing for Chalmers exit ramp from Edsel Ford Expressway eastbound in Detroit.

Figure 29. Chalmers exit ramp from Edsel Ford Expressway; volume on expressway upstream from ramp is near possible capacity.

had counts of 192, 185, 186, and 213 vehicles. During the full hour of congested operation, four counts in the 190 to 200 range were recorded for lane 2. Expansion of these 5-min counts to an hourly rate gives volume rates ranging from 2,184 to 2,556 vph/lane.

OPERATION OF A DIAMOND OFF-RAMP

A diamond off-ramp studied in Detroit was the Chalmers off-ramp from the Edsel Ford Expressway eastbound. This ramp is relatively far out on the expressway (6½ mi from the Ford-Lodge interchange) in a residential area. The expressway at the location is level and straight. The ramp exit sign (Fig. 28) could be seen when still 0.4 mi upstream from the nose of the ramp (Fig. 29), though it probably would not ordinarily be seen when that far away by the strangers most in need of it. Although the freeway volumes were near possible capacity when the pictures were taken, the volumes appear to be much lower. A tapered deceleration lane 400 ft long precedes the nose. Practically all the ramp vehicles used the complete deceleration lane, beginning the turn off lane 1 at the beginning of the taper. Speeds in lane 1 were not decreased for this maneuver as far as could be determined (average 45 mph, with lanes 2 and 3 traveling 51 to 57 mph). A number of drivers used turn signal indicators (approximately 40 percent during spot checks). The ramp exits to the service road 980 ft before reaching the cross street. During the peak hour selected, the cross street admitted by signal 1,066 vehicles, of which 447 turned left, 447 went through on the service road, and 172 turned right. The leg was 3 lanes wide. At no time was there any serious backup although the left-turn lane did have about 20 loaded cycles (60-sec cycle).

The volume obtained during the one hour of peak flow (all of which was high-speed free flow) was: ramp, 1,092 vph, 1.1 percent trucks; lane 1, 868 vph, 2.4 percent trucks; diverge, 1,960 vph (ramp + lane 1); lane 2, 1,944 vph; lane 3, 1,988 vph; freeway, 4,800 vph (lane 1 + lane 2 + lane 3); total, 5,892 vph, 0.6 percent trucks freeway + ramp); approaching average per lane, 1,963 vph; going-away average per lane 1,599 vph (82 percent in lanes 2 and 3).

The highest 5-min count obtained was a 6,540-vph rate, which is an average of 2,180 vph per approaching lane. Despite the very high volumes, this location appeared capable of handling more traffic had the demand been present. The highest 5-min lane count was 184 vehicles. The highest 5-min diverging count (ramp + lane 1) was 183 vehicles.

Despite the high hour volume recorded, the study location at no time appeared in danger of queueing or developing a backup. A distinguishing feature of the entire operation was the steady demand. Ordinarily a location carrying 1,963 vph/lane average will have short periods of such high demand that the free-flow operation cannot be sustained. As previously mentioned, the highest 5-min count averaged only 2,180 vph/lane when expanded to one hour. The fact that the expressway traffic was outbound in the evening peak at an outlying location was also an advantage, as volumes downstream from the study location were less than 1,600 vph/lane average.

The excellent performance obtained at this location suggests that perhaps attention is not being focused on the right aspects of exit ramps. The low commercial vehicle percentage, narrow angle of divergence (4°), adequate deceleration distance on the ramp, steady demand, limited weaving, and good target value of this location are all reasons for the high quality of performance. On the other hand, the deceleration lane is shorter than one might desire for a high-volume location yet it was used perfectly by the drivers. In summary, perhaps the need is simply for an exit, that can be seen and driven at normal lane 1 speeds. Other matters, such as commercial vehicles and traffic fluctuations, are not so easily controlled.

An Investigation of Some Traffic Flow Characteristics

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Some characteristics of the distribution of vehicular spot speeds and the headway characteristics of vehicles on two-lane highways were investigated with the ultimate goal that one of these characteristics might be developed into a suitable criterion for the assessment of traffic congestion. It was determined that, within the scope of the study, the standard deviation of the spot speed distribution had good indication of being a significant traffic flow parameter.

The spot speed and the headway characteristics are discussed under separate headings; a third section is devoted to a discussion of the association of spot speed and headway characteristics.

ONE of the main goals in traffic engineering is to eliminate or reduce traffic congestion. In the modern mechanized society the importance of smooth and efficient motor vehicle transportation can hardly be overemphasized. Congestion, is usually considered to exist when vehicles cannot flow freely; but what exactly constitutes free flow has not been well defined. There is no widely-accepted quantity which measures the amount of congestion on some section of roadway and which can be used for making comparisons between different sections of roadway.

Traffic density, volume of traffic, and mean speed of traffic are generally considered to be the three basic features of traffic flow (1, 2). However, none of these features alone can be conveniently used as a measure of congestion. A mere statement of volume will not suffice because a high volume at a reasonably high speed is not indicative of congestion; conversely speed alone is not a good criterion. Density, on the other hand, incorporates both volume and speed and is a better criterion than either volume or speed, although it does not specify the particular combination of the two. Another shortcoming of density is that it does not account for the nature or composition of traffic. As a hypothetical example for the latter, 30 passenger cars in a mile of roadway or 30 trailer trucks in a mile of roadway would both be expresssed as 30 vehicles per mile in terms of density. As far as the state of congestion is concerned, however, these two conditions of traffic are not equivalent.

Several studies (3, 4, 5, 6) have been conducted to develop other criteria to measure or describe congestion. All have required extensive data collection and analysis, and none can be used for the detection of congestion as it develops.

It is believed, however, that a characteristic of the traffic stream may be developed into a suitable criterion to be used in the assessment of relative congestion. As an initial investigation two-lane rural highways were considered; this study may serve as a stepping-stone for future research on higher-type facilities.

In many cases, the speed at which a driver travels on a two-lane highway is not the speed he would have set for himself commensurate with the capabilities of his vehicle, the roadway features, and environmental conditions. The presence of other vehicles on the highway forces the driver to deviate from his desired speed. As the number of vehicles on the highway increases, it can reasonably be conjectured that the

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freedom of the individual driver will be increasingly restricted. Therefore, the characteristics of the distribution of spot speeds will change as traffic density increases. Also the time spacing characteristics of vehicles may be expected to be dependent on density. The purpose of this study was to investigate the spot speed and time spacing characteristics on two-lane highways as to their variation with traffic density, with the ultimate goal that one of these characteristics might be developed into a criterion for assessing the relative congestion on such highways. Traffic density was selected as the base for the study because it is a fairly good indication of congestion—certainly better than any other.

SPOT SPEED CHARACTERISTICS

The following characteristics of spot speeds were considered:

1. Amount of skewness of the spot speed distribution.
2. Amount of kurtosis of the spot speed distribution.
3. Deviation of the observed spot speed distribution from a normal distribution as measured by the chi-square test.
4. Mean of the spot speed distribution.
5. Standard deviation of the spot speed distribution.

The skewness of a symmetrical distribution is zero; the skewness of an observed spot speed distribution can be either negative or positive, depending on the direction in which the tail of the distribution extends. Positive skewness results when the tail of the frequency curve extends more toward the higher values of the distribution than toward the lower values, and vice versa.

The kurtosis of a normal distribution has the numerical value 3. Curves more peaked than the normal are called leptokurtic and have kurtosis values greater than 3. Curves flatter than the normal curve are called platykurtic and have kurtosis values less than 3. Thus, the observed kurtosis value provides a comparison with a normal distribution.

The chi-square test indicates whether an observed distribution deviates significantly from an expected distribution. In this study a normal distribution with the same number of observations, the same mean, and the same standard deviation as the observed spot speed distribution was constructed. Subsequently, the normal and the observed distributions were compared with the chi-square test.

The mean and the standard deviation are independent parameters of a distribution. The method of computation for the mean and the standard deviation is given in the appendix, together with those for the first three characteristics.

Correlations were sought between these five characteristics and traffic density. Traffic density was taken as the average over a period of one hour in one lane. The term average lane density is used throughout this report. It was computed by

$$D = \frac{V}{S} \tag{1}$$

in which

D = average lane density, in vehicles per mile;
V = hourly directional traffic volume, in vehicles per hour; and
S = average speed, in miles per hour.

Data Collection

To compute average lane density and the five spot speed characteristics it was necessary to obtain speed data and directional volume counts. Speed data taken by a radar speed meter were recorded on a graphic recorder tape. To be able to distinguish between directions on the tape, and thus enable a directional volume count, a chronograph pen was used in conjunction with the graphic recorder. The chronograph pen was actuated manually by a telegraph key arrangement and was caused to make a "blip" in the margin of the tape as each vehicle crossed a reference mark on the pavement in the direction under study. The chronograph pen also enabled the measurement of headways; this point is discussed in a subsequent section.

Data were collected on two level and tangent sections of rural two-lane highways, virtually free from on-and-off turning traffic in the vicinity of the data collection spot. Trucks constituted about 15 percent of the traffic at both locations and both had speed limits of 55 mph, which is the absolute speed limit for such highways in Virginia. Weather conditions and visibility were favorable during data collection at both locations. Thus, all conditions that might cause variations in speed distributions between locations were in essence identical.

The study locations are referred to as location I and location II.

Results

It is generally believed that increased traffic densities cause positive skewness, leptokurticity, and deviation from normality (7). In the present study, however, none of these trends was observed. None of the three characteristics had a definite pattern of variation with traffic density. Table 1 gives observed values of skewness and kurtosis, together with observed and significant values of chi-square. No further analyses seemed warranted on the basis of the data in Table 1.

Table 2 gives values of mean speed and standard deviation for different average lane densities. An analysis indicated no significant correlation between the mean speed and the average lane density at either location. Apparently the generally accepted hypothesis that mean speed drops with increasing traffic density does not hold true for such low values of density.

The correlation between standard deviation and average lane density was significant at both locations. The coefficients of correlation were -0.535 with 9 degrees of freedom for location I and -0.736 with 5 degrees of freedom for location II (Fig. 1). These

TABLE 1

SKEWNESS, KURTOSIS AND CHI-SQUARE VALUES

Location	Average Lane Density (vpm)	Skewness	Kurtosis	Chi-Square Value		
				For 95% Significance	Deg. of Freedom	Observed
I	3.08	-0.41	3.84	22.362	13	25.117[1]
	3.87	0.20	4.51	15.507	8	14.730
	4.12	-0.01	2.46	21.026	12	30.196[1]
	4.18	0.36	4.39	19.675	11	12.564
	5.11	0.62	5.47	16.919	9	7.246
	5.47	0.30	4.04	16.919	9	38.480[1]
	5.47	0.12	4.57	21.026	12	20.748
	6.35	0.28	3.51	18.307	10	18.745[1]
	6.65	0.26	3.44	18.307	10	10.369
	8.61	-0.12	3.46	16.919	9	7.973
	9.87	0.21	4.54	18.307	10	26.325[1]
II	2.71	-0.74	4.40	21.026	12	18.561
	4.07	-0.32	2.81	19.675	11	31.759[1]
	5.00	-0.21	2.90	21.026	12	25.309[1]
	6.12	-0.52	3.62	18.307	10	26.344[1]
	6.14	-0.05	3.09	19.675	11	12.789
	7.20	-0.11	2.77	19.675	11	12.180
	8.88	-0.39	3.13	18.307	10	26.512[1]

[1] Significant at 95 percent level.

coefficients, although not very high, were nevertheless significant at the 90 percent level.

The apparent parallelism and the proximity of the two regression lines suggested the testing of the hypotheses that the slopes of the lines were equal and that the intercepts of the two lines were equal. Tests revealed that neither of these hypotheses could be rejected; therefore, the lines could be accepted as representing the same relationship. The line which fit all of the points was determined to be $\sigma = 6.876 - 0.164$ D ("composite" in Fig. 1). The coefficient of correlation in this case was -0.526 with 16 degrees of freedom, significant at the 95 percent level. Higher order curves (second, third, and fourth) were fitted to the entire group of data to improve the correlation between the standard deviation and density; however, an analysis of variance on the residual sums of squares from the higher order curves indicated that a significant improvement was not achieved.

TABLE 2

MEAN SPEED AND STANDARD DEVIATION AT DIFFERENT AVERAGE LANE DENSITIES

Location	Average Lane Density (vpm)	Speed (mph)	
		Mean	Std. Dev.
I	3.08	49.65	6.96
	3.87	51.10	5.24
	4.12	50.48	6.44
	4.18	50.03	6.21
	5.11	50.27	5.18
	5.47	50.00	5.15
	5.47	49.03	6.22
	6.35	49.45	5.36
	6.65	49.05	5.81
	8.61	49.62	4.97
	9.87	49.87	5.52
II	2.71	52.49	6.89
	4.07	52.60	6.11
	5.00	52.68	6.83
	6.12	52.72	6.33
	6.14	52.11	6.07
	7.20	51.80	5.75
	8.88	52.24	5.91

The 95 percent confidence limits of a prediction from the correlation between standard deviation and average lane density were computed (Fig. 2).

Figure 2 also shows standard deviations computed from data collected by the Virginia Department of Highways independently of the original study. The locations where these data were collected had roadway and traffic features substantially different from the two original study locations. However, it will be observed that all of the points fell within the 95 percent confidence limits. It is interesting to mention that mean speeds on the two additional locations dropped below those observed on the original locations, even in connection with low densities, because of excessive truck percentage and poor

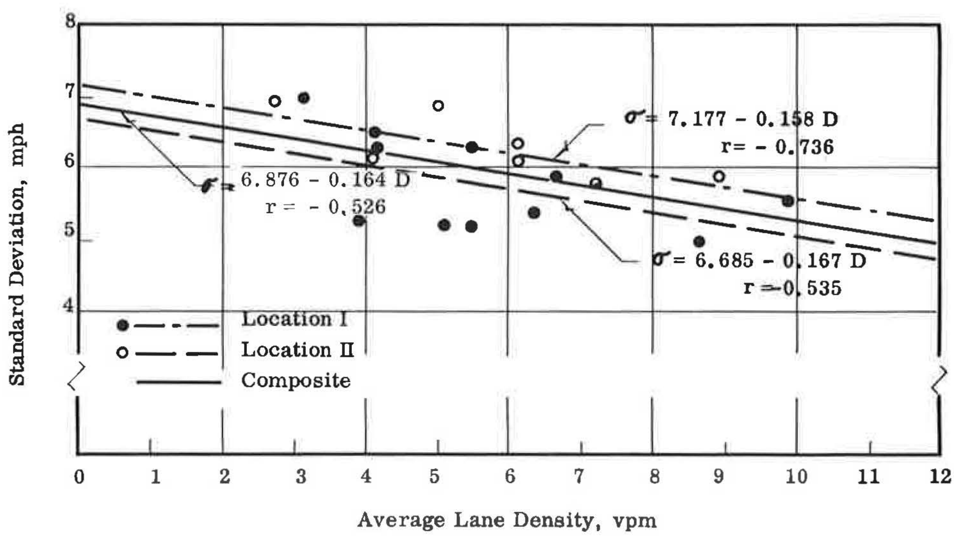

Figure 1. Standard deviation vs average lane density.

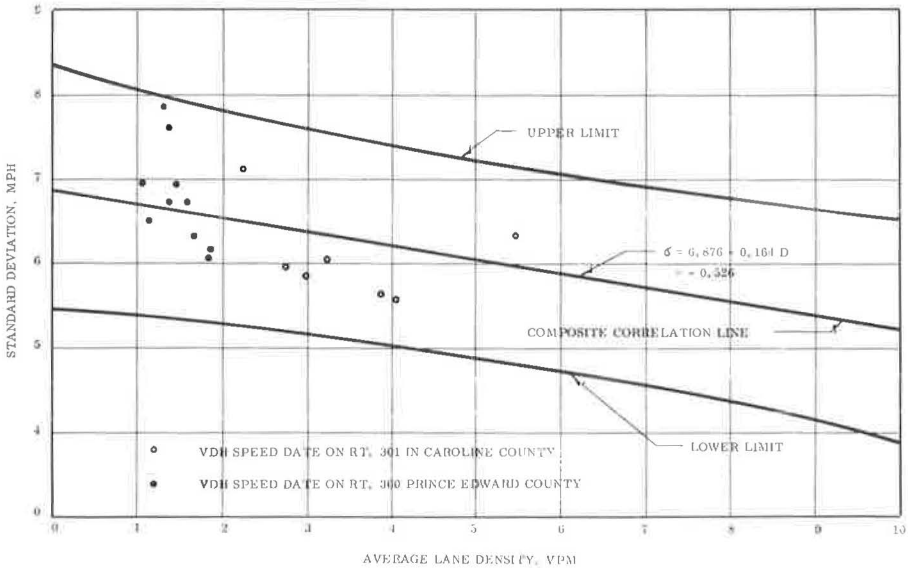

Figure 2. Correlation with 95 percent confidence limits of one observation.

vertical alignment; however, the relationship between standard deviation and density still held.

Discussion

The skewness, the kurtosis, the deviation from normality as measured by a chi-square test, and the mean of the spot speed distribution are not very significant characteristics of traffic flow within the range of densities observed. None of them can be used as a means of assessing the relative congestion on two-lane highways. The standard deviation of the spot speed distribution, on the other hand, is a significant characteristic, as suggested by the data obtained at the two original locations and later borne out by the data from two additional locations. It seems that although the other four characteristics may be influenced by some factors like purpose of trip of the driver or the physical condition of the driver, the standard deviation is free from these influences. That is to say, if it were possible to obtain a partial correlation coefficient of, for example, mean speed versus density, holding all other possible influences constant, that coefficient might be significant. In the present study those influences were altogether neglected. However, the standard deviation showed a correlation with traffic density under identical conditions.

Summary

Among the five spot speed characteristics studied the standard deviation was the only one that showed a significant correlation with traffic density. It was established that the relationships between standard deviation and density obtained from the two locations were not significantly different and that a composite regression line represented all the data better than two individual lines. Confidence limits were set on the composite regression line; it was observed that speed data collected by the Virginia Department of Highways independently of the original study at two locations with different roadway and traffic features conformed to the findings of the original study.

HEADWAY CHARACTERISTICS

The headway characteristics considered were the percentages of vehicles traveling closer than 1, 2, 3, 4, 5, 6, 7, 8, 9, and 10 seconds. These percentages were determined for all the density levels at which spot speed distributions were obtained. It is expected that the percentage of vehicles traveling closer than a specified headway will increase as the traffic density increases. Correlation analyses were run between average lane density and the percentage of vehicles traveling closer than each of the headway values stated.

Data Collection

It was mentioned earlier that the chronograph pen enabled the collection of headway data. The tape of the graphic recorder could be run at any of ten different speeds. In this study, after considering the bulk of the tapes and a satisfactory speed trace, a tape speed of 6 in. per minute was used. Because the tape speed was known, the time spacing between vehicles was derived from the distance between "blips" in the margin of the tape.

Results

In general, a high degree of correlation was attained between the percentage of vehicles traveling closer than a specified headway and average lane density. Only the 1-sec headway produced a non-significant result. Table 3 summarizes the percentage of vehicles traveling closer than each headway value. Table 4 gives the correlation coefficients obtained at both locations.

Regression lines were determined (Table 5) for the data on percentage of vehicles traveling closer than 2, 3, 4, 5, 6, 7, 8, 9, and 10 sec and density. The 1-sec headway was omitted because a non-significant correlation was observed in that case.

Figure 3 is a plot of the regression lines given in Table 5. The general parallelism of the lines suggested the testing of the hypothesis that the regression lines for the same headway from the two locations were parallel. The test results indicated that this hypothesis could not be rejected for any of the pairs of lines. Next the hypothesis was set up that the intercepts of the same pairs of lines were equal. This hypothesis, however, had to be rejected in all cases. Therefore, a generalization between the two locations was not possible; in other words, a single composite line would not represent all the points pertaining to the same headway better than two individual lines.

TABLE 3

PERCENTAGE OF VEHICLES TRAVELING CLOSER THAN SPECIFIED HEADWAY

Location	Avg. Lane Den. (vpm)	Percentage of Vehicles Traveling Closer Than									
		1 Sec	2 Sec	3 Sec	4 Sec	5 Sec	6 Sec	7 Sec	8 Sec	9 Sec	10 Sec
I	3.08	10.20	24.83	29.59	35.03	38.77	41.83	44.89	45.57	46.59	48.97
	3.87	8.59	28.12	37.89	43.75	45.70	47.26	50.00	51.95	53.90	56.25
	4.12	8.25	24.75	34.65	41.58	46.20	49.83	51.48	54.45	56.43	59.07
	4.18	6.23	25.90	35.07	41.30	45.24	47.53	50.15	52.12	55.07	56.05
	5.11	10.92	29.68	40.94	46.74	48.45	51.18	53.57	55.96	56.30	59.37
	5.47	7.37	29.81	39.42	43.27	48.72	52.56	56.73	59.29	62.82	65.06
	5.47	12.41	34.63	44.43	49.66	52.27	56.19	57.83	59.13	61.09	61.75
	6.35	5.52	28.56	40.25	47.07	51.29	53.88	56.48	58.75	62.32	65.89
	6.65	8.75	33.75	42.19	48.12	53.12	55.94	60.00	62.50	64.69	67.50
	8.61	9.96	37.36	48.74	56.93	60.13	63.33	68.31	70.45	74.00	77.56
	9.87	8.02	36.72	51.23	58.33	65.42	70.98	73.76	75.61	78.08	80.24
II	2.71	6.25	27.94	37.13	40.44	44.85	47.05	50.36	51.10	53.30	55.14
	4.07	10.90	33.01	42.63	48.40	51.60	55.13	57.37	59.29	60.90	61.86
	5.00	10.67	40.66	48.66	53.66	57.66	60.33	63.33	64.99	66.33	67.33
	6.12	10.90	38.86	48.81	55.45	60.66	64.45	67.29	69.19	71.08	74.40
	6.14	9.63	40.36	50.61	56.82	61.17	63.34	64.89	67.07	68.93	69.55
	7.20	11.43	43.78	57.17	63.05	67.63	69.26	71.22	73.51	75.14	76.77
	8.88	9.61	46.72	56.76	62.00	69.42	72.91	74.66	76.40	77.28	78.15

TABLE 4

CORRELATION OF PERCENTAGES CLOSER THAN SPECIFIED HEADWAY WITH AVERAGE LANE DENSITY

Headway (sec)	Coefficient of Correlation Between Average Lane Density and % of Vehicles Traveling Closer Than Indicated Headway	
	Location I	Location II
1	-0.055	0.048
2	0.874[1]	0.944[2]
3	0.939[1]	0.948[2]
4	0.942[1]	0.941[2]
5	0.980[1]	0.973[1]
6	0.975[1]	0.979[1]
7	0.956[1]	0.974[1]
8	0.984[1]	0.970[1]
9	0.980[1]	0.967[1]
10	0.982[1]	0.951[1]

[1] Significant at 99.9 percent level.
[2] Significant at 99 percent level.

TABLE 5

REGRESSION EQUATIONS FOR PERCENTAGE TRAVELING CLOSER THAN THE SPECIFIED HEADWAY AND AVERAGE LANE DENSITY

Headway (sec)	Regression Lines for Density and Percentage[1] Traveling Closer Than Specified Headway	
	Location I	Location II
2	P = 19.294 + 1.941 D	P = 21.879 + 2.946 D
3	P = 24.311 + 2.819 D	P = 29.718 + 3.334 D
4	P = 28.812 + 3.103 D	P = 33.554 + 3.613 D
5	P = 30.560 + 3.490 D	P = 35.481 + 4.104 D
6	P = 32.005 + 3.797 D	P = 37.974 + 4.155 D
7	P = 34.501 + 3.881 D	P = 41.676 + 3.924 D
8	P = 35.514 + 4.063 D	P = 42.577 + 4.076 D
9	P = 36.557 + 4.286 D	P = 45.038 + 3.931 D
10	P = 38.224 + 4.415 D	P = 46.778 + 3.883 D

[1] P is percentage of vehicles traveling closer than specified headway.

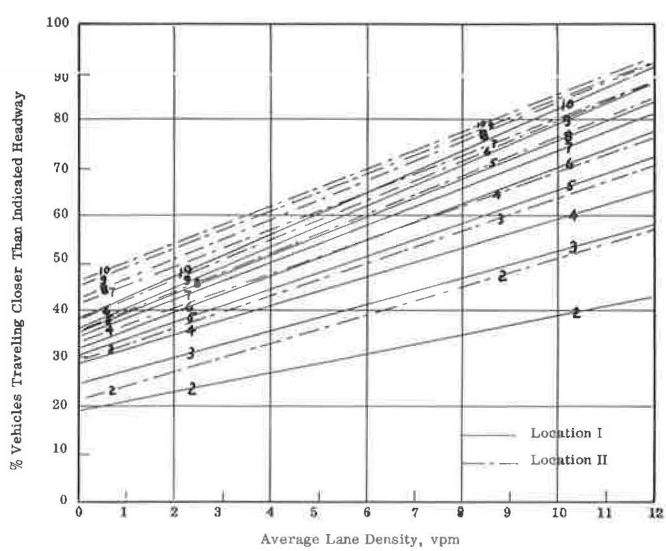

Figure 3. Regression lines for percentage of vehicles traveling closer than indicated headways and average lane density.

Discussion

Analysis of the headway frequency distributions revealed a high degree of correlation between the percentage of vehicles traveling closer than a specified headway and average lane density for headways of 2 sec and greater. The percentages of vehicles traveling closer than 1 sec did not indicate a correlation with traffic density, pointing out the fact that some drivers tend to follow a leading car very closely regardless of the prevailing traffic conditions. The percentage of vehicles traveling closer than a 1-sec headway cannot, therefore, be considered a significant characteristic of traffic flow and can have no applicability in the assessment of relative congestion. The percentages of vehicles traveling closer than headways of 2 sec or greater may be a significant characteristic of traffic flow; however, the fact that the regression lines for the percentage of vehicles traveling closer than a specified headway and average lane density from the two locations were not coincident, although parallel, cannot be overlooked. The purport is that although the rate of variation of the percentage of vehicles closer than a specified headway with density is the same for either location (i.e., the slopes of the lines are equal), there is a factor which influences the distribution of headways in a different manner at different locations. Unfortunately, headway data from other sources were not available to carry this phase of the investigation further.

Summary

Although very high correlations were obtained between average lane density and the percentage of vehicles traveling closer than headways of 2, 3, 4, 5, 6, 7, 8, 9, and 10 seconds at both locations, a generalization between locations was not possible. Headway characteristics may yet be used in assessing relative congestion if the cause or causes of variation between locations can be identified.

ASSOCIATION OF SPOT SPEED AND HEADWAY CHARACTERISTICS

It may be expected that the spot speed characteristics and the headway charactertics discussed in the two previous sections will have a relationship; i.e., as vehicles travel with smaller headways their speeds tend to be more uniform. To investigate

TABLE 6

STANDARD DEVIATIONS AND CORRESPONDING PERCENTAGES OF VEHICLES TRAVELING CLOSER THAN SPECIFIED HEADWAYS

Location	Std. Dev. (mph)	Percentage of Vehicles Traveling Closer Than								
		2 Sec	3 Sec	4 Sec	5 Sec	6 Sec	7 Sec	8 Sec	9 Sec	10 Sec
I	4.97	37.36	48.74	56.93	60.13	63.33	68.31	70.45	74.00	77.66
	5.15	29.81	39.42	43.27	48.72	52.56	56.73	59.29	62.82	65.06
	5.18	29.68	40.94	46.74	48.45	51.18	53.57	55.96	56.30	59.37
	5.24	28.12	37.89	43.75	45.70	47.26	50.00	51.95	53.90	56.25
	5.36	28.56	40.25	47.07	51.29	53.88	56.48	58.75	62.32	65.89
	5.52	36.72	51.23	58.33	65.42	70.98	73.76	75.61	78.08	80.24
	5.81	33.75	42.19	48.12	53.12	55.94	60.00	62.50	64.69	67.50
	6.21	25.90	35.07	41.30	45.24	47.53	50.15	52.12	55.07	56.05
	6.22	34.63	44.43	49.66	52.27	56.19	57.83	59.13	61.09	61.75
	6.44	24.75	34.65	41.58	46.20	49.83	51.48	54.45	56.43	59.07
	6.96	24.83	29.59	35.03	38.77	41.83	44.89	45.57	46.59	48.97
II	5.75	43.78	57.17	63.05	67.63	68.26	71.22	73.51	75.14	76.77
	5.91	46.72	56.76	62.00	69.42	73.91	74.66	76.40	77.28	78.15
	6.07	40.36	50.61	56.82	61.17	63.34	64.89	67.07	68.93	69.55
	6.11	33.01	42.63	48.40	51.60	55.13	57.37	59.29	60.90	61.86
	6.33	38.86	48.81	55.45	60.66	64.45	67.29	69.19	71.08	74.40
	6.83	40.66	48.66	53.66	57.66	60.33	63.33	64.99	66.33	67.33
	6.89	27.94	37.13	40.44	44.85	47.05	50.36	51.10	53.30	55.14

the relationship between these characteristics, correlation analyses were run between the standard deviation of the spot speed distribution and the percentage of vehicles traveling closer than 2, 3, 4, 5, 6, 7, 8, 9, and 10 sec. In these correlation analyses each point represented the standard deviation and the percentage of vehicles traveling closer than the specified headway pertaining to the same density level.

Results

Table 6 gives the standard deviations and the corresponding percentages of vehicles traveling closer than the specified headways.

Table 7 summarizes the results of the regression analysis on data for the standard deviation and on percentage closer than a specified headway. In general, a very high degree of correlation does not exist between the percentage of vehicles traveling closer than a specified headway; however, an overall trend is apparent.

Corresponding lines from each location were compared for parallelism and coincidence. Hypotheses were set up that the slopes and the intercepts of each pair of lines were equal. It was determined that in each case, with the given scatter of points, the hypothesis that the slopes of the two lines were equal could not be rejected. However, the hypothesis that the intercepts were equal had to be rejected. Therefore, for any pair it was impossible to draw a single line that would represent all the points better than two separate lines.

Figure 4 is a plot of the percentage of vehicles traveling closer than 3 sec against the standard deviation for both locations; this plot is representative of those with other values of headway.

Discussion

The results of the attempt to associate the standard deviation of the speed distribution to the percentage of vehicles traveling closer than a specified headway were not very encouraging. The correlation obtained between these two quantities did not reach a high level and in certain instances was below the 90 percent significance level (Table 6). At those values of headway where the correlation was significant, the regression lines from the two locations displayed parallelism in all cases, but none of the pairs was coincident. The effect which was observed to cause the non-coincidence in the case of the regression lines for the percentage of vehicles traveling closer than a

TABLE 7

SUMMARY OF REGRESSION ANALYSES ON DATA FOR STANDARD DEVIATION AND PERCENTAGE OF VEHICLES CLOSER THAN A SPECIFIED HEADWAY

Headway (sec)	Coeff. of Correl. Between Std. Dev. and Percent Closer Than Specified Headway		Line of Best Fit for Data on Std. Dev. and Percent Closer Than Specified Headway	
	Location I	Location II	Location I	Location II
2	-0.500	-0.622	—[1]	—[1]
3	-0.618[2]	-0.731[3]	$P = 74.892 - 6.010\,\sigma$	$P = 123.697 - 11.943\,\sigma$
4	-0.594[3]	-0.758[2]	$P = 82.507 - 6.276\,\sigma$	$P = 139.076 - 13.539\,\sigma$
5	-0.533[3]	-0.741[3]	$P = 85.410 - 6.092\,\sigma$	$P = 150.015 - 14.518\,\sigma$
6	-0.479	-0.737[3]	—	$P = 152.830 - 14.532\,\sigma$
7	-0.518	-0.704[3]	—	$P = 146.660 - 13.160\,\sigma$
8	-0.546[3]	-0.721[3]	$P = 100.133 - 7.227\,\sigma$	$P = 154.114 - 14.066\,\sigma$
9	-0.559[3]	-0.725[3]	$P = 106.043 - 7.852\,\sigma$	$P = 153.416 - 13.694\,\sigma$
10	-0.593[3]	-0.701[3]	$P = 112.430 - 8.547\,\sigma$	$P = 152.356 - 13.292\,\sigma$

[1] Line of best fit not calculated because correlation not significant. [2] Correlation significant at 95 percent level, but not at 98 percent level. [3] Correlation significant at 90 percent level, but not at 95 percent level.

lation developed in the original study between the standard deviation and average lane density.

3. Point 2 accentuates the indication that for two-lane highways in rural areas the standard deviation of the speed distribution is a significant parameter of traffic flow and may be used in assessing congestion.

4. The percentage of vehicles traveling closer than a headway of 1 sec did not indicate a correlation with average lane density. The percentage for headways of 2 sec and greater, on the other hand, were correlated with average lane density to a high degree of significance at both locations. However, the regression lines for the two locations could not be combined to obtain a composite single line. This prevented the possibility of a generalization. Headway data were not available from other sources to investigate this possibility further.

5. It was determined that the percentage of vehicles traveling closer than a specified headway was not significantly correlated with the standard deviation for certain values of the headway, that even for those values where the correlation was significant for both locations a generalization was impossible, and that increasing interference between vehicles tends to make speeds more uniform.

6. The standard deviation seems to be a significant parameter of traffic flow—better than the mean speed and the percentage of vehicles traveling closer than a specified headway—at least in the range of densities studied on two-lane rural highways. In assessing congestion the standard deviation should be the best parameter to specify because indications are that it may be applicable to many kinds of roadway and traffic conditions. Furthermore, the standard deviation of a speed frequency distribution can be readily estimated.

ACKNOWLEDGMENTS

The results of this study, conducted under the sponsorship of the Virginia Council of Highway Investigation and Research, were presented to the University of Virginia as a thesis in partial fulfillment for the degree of Master of Civil Engineering. The author wishes to acknowledge the assistance and guidance given by T. E. Shelburne, Director of Research of the Council, and by H. G. Larew, C. E. Echols, and C. E. Schwartz, of the University of Virginia faculty. The help of J. D. Decker, formerly head of the Council's Traffic and Safety Section, in the initial formulation of the proect and the development of the instrumentation has proven invaluable. Finally the author wishes to extend thanks to F. P. Nichols and other staff members of the Virginia Council of Highway Investigation and Research who assisted in the study.

REFERENCES

1. Greenshields, B. D., "The Density Factor in Traffic Flow." Traffic Eng. p. 26 (March 1960).
2. May, A. D., and Wagner, F. A., "Headway Characteristics and Interrelationships of the Fundamental Characteristics of Traffic Flow." HRB Proc. 39: 524 (1960).
3. Rothrock, C. A., and Keefer, L. E., "Measurement of Urban Traffic Congestion." HRB Bull. 156, pp. 1-13 (1957).
4. Wingo, L, "Measurement of Congestion in Transportation Systems." HRB Bull. 221, pp. 1-28 (1959).
5. Deen, T. B., "The Use of Travel Time as a Factor in Rating Urban Streets." Traffic Eng., p. 13 (Jan. 1960).
6. Brenner, R., Telford, E. T., and Frischer, D., "A Quantitative Evaluation of Traffic in a Complex Freeway Network." HRB Bull. 291, pp. 163-206 (1961).

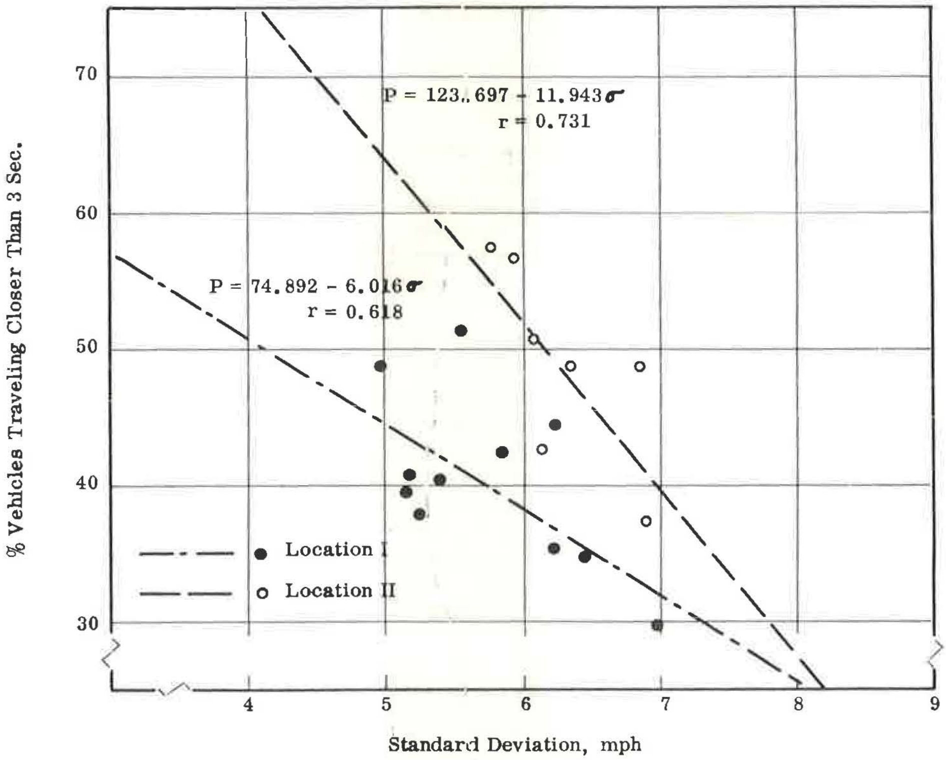

Figure 4. Percentage of vehicles traveling closer than 3 sec versus the standard deviation of the speed frequency distributions.

specified headway and density, apparently influenced this correlation and made a generalization impossible. However, the overall trend of decreasing percentage of vehicles traveling closer than a specified headway with increasing standard deviation, or the converse, indicates that increasing interference between vehicles, will tend to cause increasingly uniform speeds.

Summary

The relationship between the standard deviation of the spot speed distribution and the percentage of vehicles traveling closer than a specified headway is not well defined, although an overall trend is apparent. A generalization of this relationship between the two locations is not possible. The fact that no significant correlations were obtained for certain values of headway does not allow any positive statements.

GENERAL CONCLUSIONS

1. A correlation was not found between average lane density and certain characteristics of the spot speed distribution of vehicles (namely, skewness, kurtosis, deviation from normality, and mean speed) in the range of densities studied; on the other hand, the standard deviation of the distribution correlated with average lane density at each location. Further it was determined that the two regression lines for the standard deviation and density from the two locations could be replaced by one composite line, thus opening up possibilities of generalization.

2. Speed data obtained by the Virginia Department of Highways independent of this study and at two locations having roadway and traffic features quite different from those of the two original locations were analyzed. The computed standard deviation values were seen to fall within the 95 percent confidence range of a prediction from the corre-

Appendix

SAMPLE COMPUTATION FOR MOMENT ANALYSIS AND χ^2 TEST

Given: The speed frequency distribution of Table 8 (at location I).

TABLE 8

SPEED FREQUENCY DISTRIBUTION OBTAINED IN 1 HOUR AT LOCATION I

Speed Class	No. in Class	Speed Class	No. in Class
32 - 33.9	1	52 - 53.9	38
34 - 35.9	3	54 - 55.9	25
36 - 37.9	5	56 - 57.9	16
38 - 39.9	6	58 - 59.9	5
40 - 41.9	14	60 - 61.9	9
42 - 43.9	32	62 - 63.9	2
44 - 45.9	41	64 - 65.9	1
46 - 47.9	43	66 - 67.9	1
48 - 49.9	43	68 - 69.9	0
50 - 51.9	40	70 - 71.9	1

Determine: Mean, speed, skewness, kurtosis, standard deviation, χ^2 value (for deviation from a normal distribution with the same mean and standard deviation); also volume and average lane density to which the distribution pertains.

A mean value of 49.00 mph is assumed and the deviation of the mid-value of each class from the assumed mean is expressed in terms of classes (Table 9).

TABLE 9

DEVIATION OF MID-VALUE OF CLASS FROM ASSUMED MEAN SPEED

Speed Class	Deviation from Mean, d	Number in Class, f	fd	fd^2	fd^3	fd^4
32 - 33.9	-8	1	- 8	64	- 512	4,096
34 - 35.9	-7	3	-21	147	-1,029	7,203
36 - 37.9	-6	5	-30	188	-1,080	6,480
38 - 39.9	-5	6	-30	150	- 750	3,750
40 - 41.9	-4	14	-56	224	- 396	3,584
42 - 43.9	-3	32	-96	288	- 864	2,592
44 - 45.9	-2	41	-84	168	- 336	672
46 - 47.9	-1	43	-43	43	- 43	43
48 - 49.9	0	43	0	0	0	0
50 - 51.9	1	40	40	40	40	40
52 - 53.9	2	38	76	152	304	608
54 - 55.9	3	25	75	225	675	2,025
56 - 57.9	4	16	64	256	1,024	4,096
58 - 59.9	5	5	25	125	625	3,125
60 - 61.9	6	9	54	324	1,944	11,664
62 - 63.9	7	2	14	98	686	4,802
64 - 65.9	8	1	8	64	512	4,096
66 - 67.9	9	1	9	81	729	6,561
68 - 69.9	10	0	0	0	0	0
70 - 71.9	11	1	11	121	1,331	14,640

The terms that have significance in the following computation are defined as follows:

σ = standard deviation, mph;
a_3 = skewness;
a_4 = kurtosis;

$\Sigma f = 326$; $\Sigma fd = 8$; $\Sigma fd^2 = 2,750$; $\Sigma fd^3 = 2,309$; $\Sigma fd^4 = 80,078$; $M'_1 = \Sigma fd/\Sigma f = 0.02454$; $M'^2_1 = 0.00060$; $M'^3_1 = 0.00001$; $M'^4_1 = 0.00000$; $M'^2 = \Sigma fd^2/\Sigma f = 8.43558$; $M'_1 M'_2 = 0.20701$; $M'^2_1 M'_2 = 0.00508$; $M'_3 = \Sigma fd^3/\Sigma f = 7.23926$; $M'_1 M'_3 = 0.17381$; $M'_4 = \Sigma fd^4/\Sigma f = 245.63804$;

$$\underline{\underline{S}} = S_0 + i M'i = 49.00000 + 2 \times 0.02454 = \underline{\underline{49.04908}}$$

$M'_2 = M'_2 - (M'_1)^2 = 8.43498$; $M'_2 = 2.90430$;

$$\underline{\underline{\sigma}} = i\sqrt{M'_2} = 2 \times 2.90430 = 5.80860$$

$(M'_2)^{3/2} = 24.49771$; $(M'_2)^2 = 71.14870$; $M'_{33} = M'_3 - 3M'_2 M'_1 + 2(M'_1)^3 = 6.46481$; $M'_4 = M'_4 - 4M'_3 M'_1 + 6(M'_1)^2 M'_2 - 3(M'_1)^4 = 244.97328$;

$\underline{\underline{a_3}} = \dfrac{M'_3}{(M'_2)^{3/2}} = 0.26377$; $\underline{\underline{a_4}} = \dfrac{M'_4}{(M'_2)^2} = 3.44312$; $\underline{\underline{V}} = \dfrac{\Sigma f}{k} \times 6 = 326$ vph; $V = f = 326$; $\underline{S} = 49.05$ mph; $\underline{\underline{D}} = \dfrac{326}{49.05} = 6.65$ vpm.

Part I of the χ^2 test is mostly self-explanatory. Col. 4 is the probability of observing the specified or a greater speed if the distribution were normal. Col. 5 is the difference in successive probabilities in Col. 4; i.e., the probability of each group. Col. 6 is obtained by multiplying the probabilities in Col. b by Σf. Because the χ^2 test introduces a bias when the expected number in a class is less than 5, the tails of the expected distribution are added up until a value greater than 5 is obtained.

In part II of the χ^2 test each value in Col. 5 is multiplied by the corresponding value in Col. 4, and the products are summed. The sum is the χ^2 value. The degrees of freedom can be computed by subtracting three from the number of items contributing to the χ^2 value; i.e., the number of rows in part II of the χ^2 test. There are 13 rows; therefore, the degrees of freedom are 10, 3 degrees of freedom being lost because the original and fitted data are made to agree as to total number, mean, and standard deviation. The χ^2 value was not significant in this case.

χ^2 TEST (PART I)

S = 49.04908 mph σ = 5.80860 mph

(1) Lower Limit of Class	(2) Dev. from Mean	(3) Col. 2 / Std. Dev. (z)	(4) P(z)	(5) $P(z_1) - P(z_2)$	(6) Expected Number in Class
- ∞	+ ∞	+ ∞	1.0000	0.0017	0.5542
32	17.0491	2.93	0.9983	0.0031	1.0106
34	15.0491	2.59	0.9952	0.0074	2.4124
36	13.0491	2.25	0.9878	0.0165	5.3790
38	11.0491	1.90	0.9713	0.0307	10.0082
40	9.0491	1.56	0.9406	0.0537	17.5062
42	7.0491	1.21	0.8863	0.0791	25.7866
44	5.0491	0.87	0.8078	0.1093	35.6318
46	3.0491	0.52	0.6985	0.1271	41.4346
48	1.0491	0.18	0.5714	0.1350	44.0100
50	- 0.9509	-0.16	0.4364	0.1314	42.8364
52	- 2.9509	-0.51	0.3050	0.1073	34.9798
54	- 4.9509	-0.85	0.1977	0.0826	26.9276
56	- 6.9509	-1.20	0.1151	0.0533	17.3758
58	- 8.9509	-1.54	0.0618	0.0324	10.5624
60	-10.9509	-1.89	0.0294	0.0165	5.3790
62	-12.9509	-2.23	0.0129	0.0078	2.5428
64	-14.9509	-2.57	0.0051	0.0033	1.0758
66	-16.9509	-2.92	0.0018	0.0012	0.3912
68	-18.9509	-3.26	0.0006	0.0004	0.1304
70	-20.9509	-3.61	0.0002	0.0002	0.0652
72	-22.9509	-3.95	0.0000	0.0000	0.0000
+ ∞	- ∞	- ∞	0.0000		

χ^2 TEST (PART II)

(1) Speed Class	(2) Expected Number	(3) Observed Number	(4) Exp. - Obs.	(5) (Exp. - Obs.) / Exp.
38	9.3562	9	0.3562	0.0381
38 - 40	10.0082	6	4.0082	0.4005
40 - 42	17.5062	14	3.5062	0.2003
42 - 44	25.7866	32	6.2134	0.2410
44 - 46	35.6318	41	5.3682	0.1507
46 - 48	41.4346	43	1.5654	0.0378
48 - 50	44.0100	43	1.0100	0.0229
50 - 52	42.8364	40	2.8364	0.0662
52 - 54	34.9798	38	3.0202	0.0863
54 - 56	26.9276	25	1.9276	0.0716
56 - 58	17.3758	16	1.3758	0.0792
58 - 60	10.5624	5	5.5624	0.5266
60	9.5844	14	4.4166	0.4607

χ^2 = 10.369 (observed), D.F. = 10; χ^2 = 18.307 (0.05, 10)

A Study of Four-Way Stop Intersection Capacities

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This study was an endeavor to determine capacities of four-way stop-controlled intersections under various traffic and operating conditions.

Three right-angle intersections in the Chicago Metropolitan Area were observed and analyzed. The average headway of departure was obtained, and the effects on capacity of left- and right-turning vehicles, the number of lanes on the cross-street, and the split in volumes between the intersecting streets, were determined. The effect of commercial vehicles was tentatively investigated, but the small size of the sample precluded any specific conclusions. Only daytime driving characteristics were observed.

Due to the amount and complexity of the data to be obtained, all intersections but one were filmed for a period of 80 min each. The movie camera technique had the advantage of requiring the least amount of field observations, and presenting a permanent record of the complex interrelationship of the data to be analyzed.

Some of the interesting findings of this study are as follows:

1. Variations in the split of volume between the two intersecting streets of a four-way stop intersection produce a significantly different headway of departure for two different intersections.
2. Left-turning vehicles have no effect on the capacity, the average headway of departure for left-turning and through vehicles not being significantly different under various traffic conditions.
3. For each 1 percent of right-turning vehicles, the capacity is increased by 0.2 percent.
4. Under pressurized and ideal traffic conditions through passenger cars per lane may be expected to be discharged across a two-lane cross-street at an average of one every 7.65 sec if the split is 50/50, and one every 7.15 sec if it is 60/40. These rates are averages for the whole intersection.
5. If the split becomes 100/0 (i.e., all on-coming vehicles are on two opposite approaches only) and for the same conditions as in item 4, one might expect a discharge rate of one vehicle every 4.05 sec from each of the two approaches.
6. For the conditions of item 4 and a 50/50 split, the capacity per lane averages one vehicle every 8.08 sec if the street to be crossed has four moving lanes.
7. Seventy percent of vehicles are found to be moving two abreast if there are two lanes on a loaded approach.

•OF ALL the problems of interest to the traffic engineer, the urban intersection at grade is undoubtedly the most important. If one considers that approximately one-half of all urban accidents and more than three-quarters of all urban delays are caused by or related to urban intersections, the range and far-reaching consequences of the problem are more fully understood.

To provide efficiency and safety of movement through these intersections, vehicles and pedestrians are regulated by various types of traffic control devices. In many cases the actual warrants used for the application of these devices need much refinement and development and are often subjects of controversy. The application of the four-way stop type of control is presently very controversial, and a definite solution

Paper sponsored by Committee on Highway Capacity.

has yet to be attained. Contradictory statements such as the following have become common language: "... the four-way stop as a solution should be used more frequently" (4); and "... there can be no logical warrant for a four-way stop except as a safety measure or as a device to satisfy pressure groups that demand action at an intersection warranting no action." (12)

Little factual study had been made in the past to crystallize the use warrants for the four-way stop intersection control. McEachern (13) reported: "While most cities do use warrants for the establishment of four-way stop intersections, the warrants are not specific; and the single most widely used warrant is the high-accident frequency at two-way stop intersections." The four-way stop, as usually employed, finds applications at urban intersections as a safety measure, or as an intermediate treatment between the two-way stop and the signal control.

In the "Manual on Uniform Traffic Control Devices" (3) the warrants for four-way stops require a total vehicular volume of 500 vph for any 8 hr of an average day, with at least 200 vehicles and pedestrians entering from the minor street. No mention is made of maximum permissible volumes. However, the Institute of Traffic Engineers suggests a maximum volume of 1,000 vph (average for 6 hr), with at least 250 vph from the side street.

These requirements are useful as a guide, but are not based on such fundamental characteristics of vehicular flow as arrival rates, departure headways, and effects of opposing and intersecting flows. A rational analysis is needed of the various relationships between these fundamental traffic features at four-way stop intersections. This, together with the subsequent derivation of capacities, is dealt with herein. It is hoped that the findings of this study will contribute to the understanding of this complex urban traffic problem.

PURPOSE AND SCOPE

The purpose of this study was to determine the basic and practical capacities of four-way stop intersections, under various geometric and traffic conditions. Capacities were derived from the average departure headway of vehicles as they enter the intersection area.

The Highway Capacity Manual (1) defines basic capacity as the "maximum number of passenger cars that can pass a given point on a lane or roadway during one hour under the most nearly ideal roadway and traffic conditions which can possibly be attained." To satisfy this definition, the headways of departure were obtained and recorded separately for passenger cars, commercial vehicles, left- and right-turning vehicles, and through vehicles. Possible capacity, which is the capacity under the prevailing roadway and traffic conditions, was derived from the basic capacity by applying certain reducing factors. The investigation and determination of these factors is included hereinafter.

Practical capacity is defined as "the maximum number of vehicles that can pass a given point on a roadway or in a designated lane during one hour without the traffic density being so great as to cause unreasonable delay, hazard, or restriction to the driver's freedom to maneuver under the prevailing roadway and traffic conditions." (1) This level of capacity is equal to the possible capacity multiplied by a factor which is given in the Manual as 0.80.

Another approach to the determination of the practical capacity of an intersection could be based on a delay criterion. Assuming random distribution of the vehicles on the approach, it is possible, using the observed average headway of departure, to determine the vehicular volume that will cause a certain percentage of the drivers to be delayed by a given preferred amount of time. Any volume that causes a greater percentage of the drivers to be delayed by the same amount of time, or causes the same percentage of drivers to be delayed by a larger amount of time, will be above practical capacity. Figure 10 is a set of curves derived for various time periods and percentages of drivers.

Variables Studied

The following variables were investigated as to their effect on the capacity of four-way stop intersections:

1. Two splits in volume (51.5/48.5 and 64.0/36.0) for one-lane approaches, during pressurized conditions, with a continuous backlog of waiting vehicles.
2. Four-lane and two-lane two-way cross-streets for similar traffic conditions.
3. Variations in left and right turns, and percentage of commercial vehicles. Unfortunately, truck travel at the intersections studied was almost negligible, and the data obtained regarding their effect on headways were insufficient.
4. Waiting vehicles on the cross-street, vs no vehicles on the cross-street, vs vehicles from cross-street entering the intersection in turns with the approach vehicles under study. These analyses are further detailed hereafter.

Limitations of the Study

The study was made at three outlying intersections in the Chicago metropolitan area, and is therefore representative of Chicago drivers only. This had the advantage of eliminating the variations resulting from dissimilar driving behavior inherent to different groups of drivers. No attempt was made to relate the results with those of other parts of the country, and nighttime driving characteristics were not investigated. All control signs bore the "4-way" supplementary message (Fig. 1), because it was believed that this signing policy insured higher capacities (2).

PROCEDURE

In consideration of the limited manpower and time available for field observations, it seemed logical to use the movie-camera technique of study. This method, besides being economical in manpower, provided a permanent record of the behavior of each traffic stream and the relationships between the streams, and allowed for leisurely extraction of the data, with possible re-running of the films whenever desired. The locations were intentionally selected to provide a suitable vantage point for filming. Figure 2 shows the camera installed for filming at a high point at the intersection of Winnetka and Hibbard Streets in Winnetka. It was at a height of approximately 15 ft, and 120 ft from the intersection.

The camera was operated by a small 100-rpm synchronous motor using 120-v ac power provided by an ordinary 12-v dc battery, through a dc-ac converter. Filming was performed at the rate of 100 frames per minute; one complete 100-ft film lasted approximately 40 min.

Two right-angle intersections were filmed for 80 min each. A third intersection was filmed for 40 min. The camera was so located as to offer a view of the whole intersection and a certain length of each leg. The headways of departure for vehicles in each lane of each approach were recorded separately for each item of study. Extraction of data, although simple, was found to be very time consuming.

Figure 1. 4-Way supplementary message on stop signs.

LOCATIONS STUDIED

Intersection A: Willow and Hibbard (Fig. 3)

The first intersection studied was Willow and Hibbard in Winnetka, a community in the outlying north suburbs of Chicago. This location was selected because of its high volume, nearly equal split, high percentage of turns and little small pedestrian and roadside interference. During the period of study (p.m. peak hour), the traffic conditions were as follows:

Date: Tuesday, April 3, 1962
Time: 4:00 to 4:45 p.m.
Total intersection volume: 1,209 vph
Split: 51.5/48.5
C.V.: 6.5%
Total intersection left turns: 23.9%
Total intersection right turns: 22.9%
Basic number of lanes: 2 on each street

Figure 2. High camera location.

Because of the sparse development of the immediate surroundings, the intersection was ideally free of pedestrians, parking, and driveway interference. In fact, not a single pedestrian was encountered during the whole study period. The effects of nearby intersection controls were minimized because of their distance (more than 2,000 ft). All sight distances were adequate.

Intersection B: Winnetka and Hibbard (Fig. 4)

Immediately south of Intersection A, in the same community, is the intersection of Winnetka and Hibbard Streets. Although it does not offer a very high volume of traffic, which is a desirable feature for the purpose of the study, the main advantage of this intersection is its different split in traffic volume. The traffic conditions during the study period were as follows:

Date: Friday, April 13, 1962

Time: 4:00 to 5:30 p.m.
Total intersection volume: 742 vph
Split: 64.0/36.0
C.V.: 3.3%
Total intersection left turns: 16.6%
Total intersection right turns: 18.8%
Basic number of lanes: 2 on each street

This location also has a high vehicle turning movement, and almost no interference from pedestrians, parking, or driveways. In contrast to Intersection A, the sight distances were adequate in three quadrants only, and very limited in the fourth. Basically, the two intersecting streets have one moving lane in each direction.

Intersection C: Cumberland and Devon (Fig. 5)

The third intersection is quite different from the previous ones. It was selected so as to provide data on headways of vehicles on a two-lane highway when crossing a street with four moving lanes. Basically, the two intersecting streets were designed for four moving lanes each, but parking reduced to two the number of moving lanes on Devon. West of the intersection, Devon Street has a 35-ft median; east of the intersection the median width is reduced to 14 ft. Cumberland Street has a 4-ft median south of the intersection only. Because the intersection is located in a well-developed

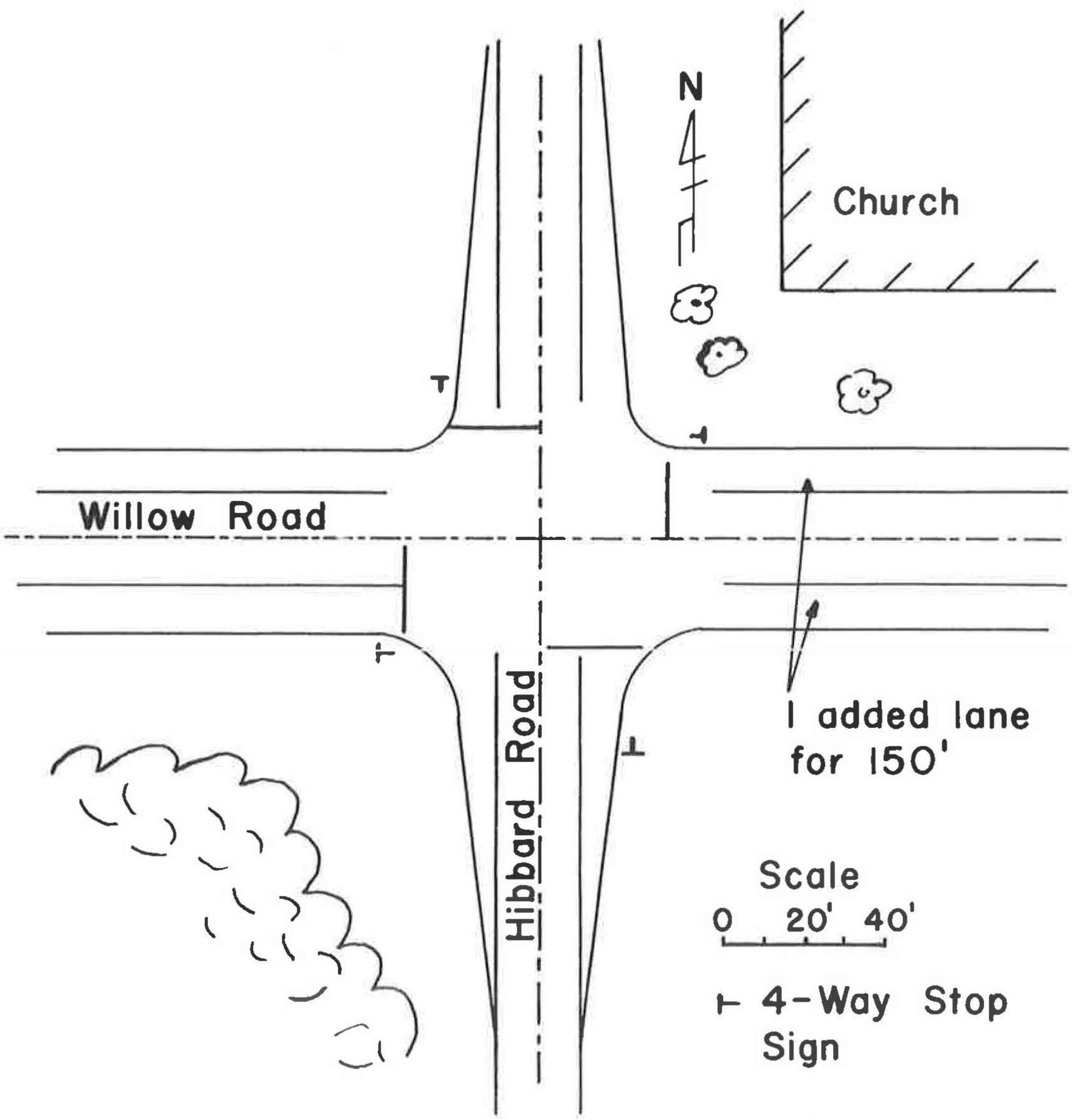

Figure 3. Intersection A (Willow and Hibbard).

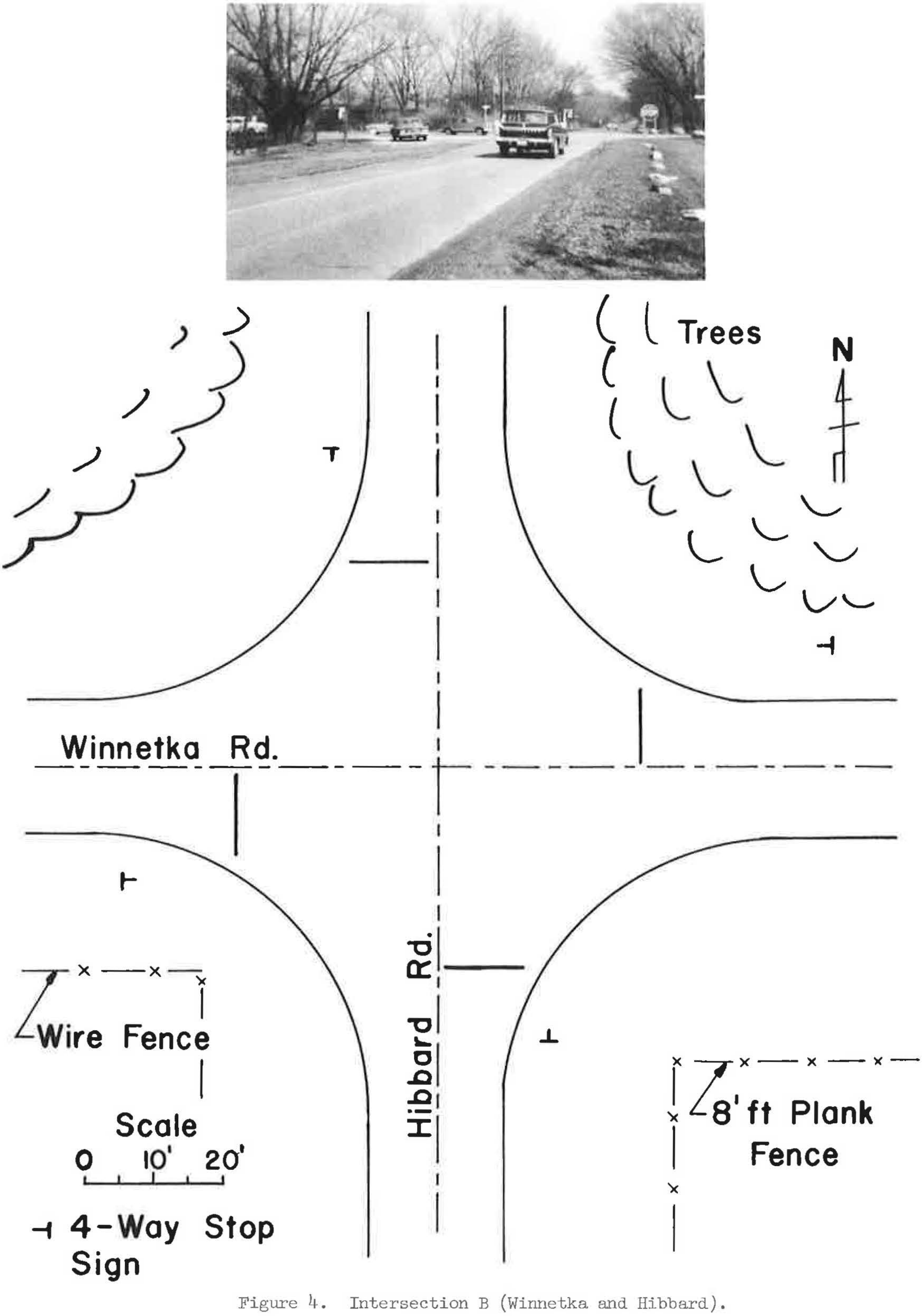

Figure 4. Intersection B (Winnetka and Hibbard).

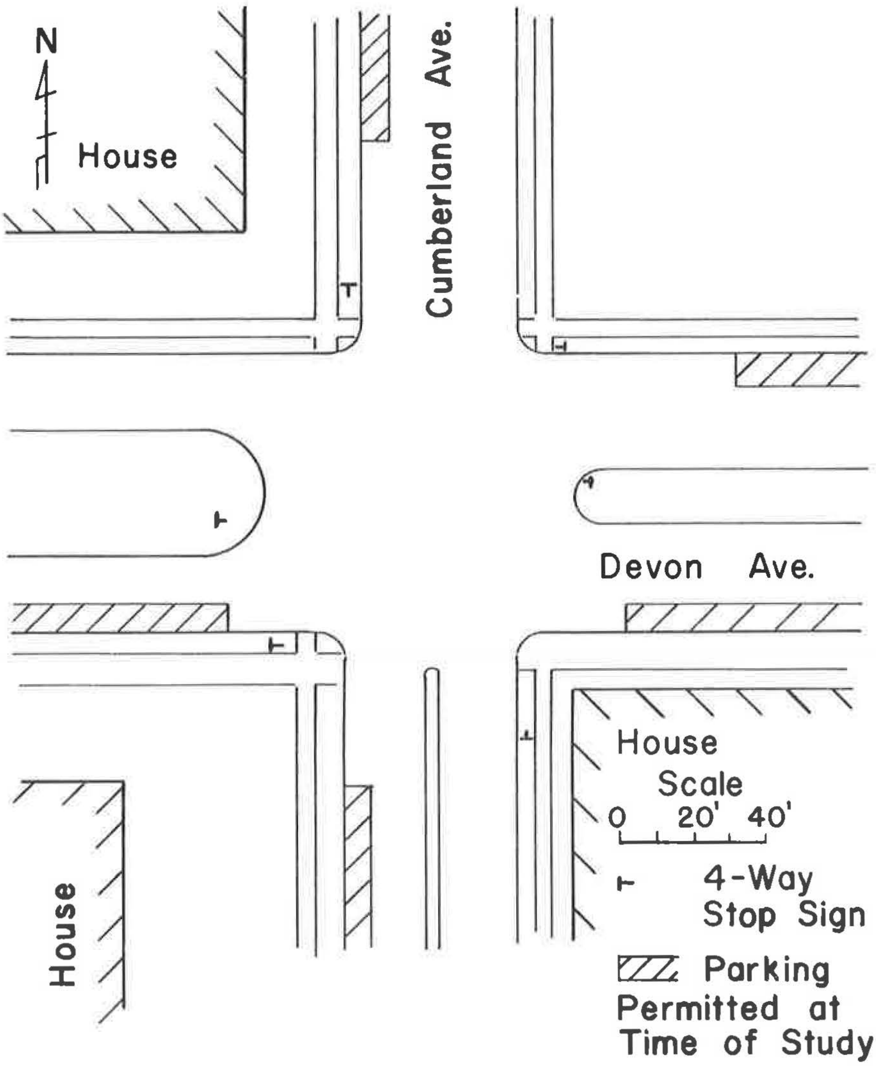

Figure 5. Intersection C (Cumberland and Devon).

community (Park Ridge), a certain amount of interference was encountered, including some pedestrian movement and parking. At the time of the study, parking was as shown in Figure 5. The eastbound traffic on Devon was studied as it crossed four-lane Cumberland Street. During the p.m. peak hour of filming, the following conditions were encountered:

Date: Friday, April 20, 1962

Time: 3:45 to 5:15 p.m.	C.V.: 4.3%
Total intersection volume: 1,800 vph	Number of lanes: as shown in Figure 3
Split: 51.2/48.8	

STATISTICAL ANALYSIS

Calculation of Minimum Sample Size

The size of a sample of data needed to give a mean within some desired range of accuracy is dependent on the desired confidence level and the standard deviation of the population. If no data are available, the standard deviation of the population may be assumed. However, if data have already been taken, an estimate of the standard deviation of the population may be obtained from that of the sample, and it becomes possible to determine whether the size of the available data is sufficiently large. Inasmuch as the data of this study were obtained from films, it was not feasible to measure time closer than one frame (0.60 sec). Using a 95 percent confidence limit, a standard deviation of 3.7, 3.0 and 2.0 frames (all sample standard deviations are smaller than 3.7 frames), and a limit of error of 1.0 frame, the size of sample needed is

$$E = 1.96 \frac{S}{\sqrt{n - 1}} \tag{1}$$

from which $n = \dfrac{3.84 \times (3.7^2,\ 3.0^2,\ 2.0^2)}{E} + 1 = 53.5,\ 35.6,\ 15.4.$

Interpreting this result, any sample of data of size 54, 36, or 16 or more, would have a mean value within 1.0 frame from the true population mean in 95 cases out of 100, depending on the consistency of the sample, as expressed in the standard deviation. These sample sizes are strict minimums, and more readings should be obtained where feasible.

Tests on Data

To calculate the basic capacity of four-way stop intersections, it is necessary to register the headway of departure of the vehicles on any approach during loaded conditions. If both streets are loaded, the vehicles usually proceed through the intersection in turns, each one moving to the first position as the cross-street vehicle accelerates. However, it often happens that one approach is clogged with waiting vehicles while there are none or few on the cross-street. Obviously, one would expect the headways to be much smaller on one street if there are no vehicles coming from the cross-street. Besides being recorded separately for passenger cars, commercial vehicles, through, left-turning, and right-turning vehicles, headways were also obtained separately for the three following cases:

1. L headways: When both streets are loaded vehicles proceed through in turns, with one vehicle accelerating from a cross-street approach within the headway recorded.
2. N headways: The approach under study is loaded, with no vehicles approaching on the cross-street (50 ft or less) or waiting at the stop line.
3. I headways: The approach under study is loaded, with interference from vehicles on the cross-street (within 50 ft of the stop line). This type of headway is there-

TABLE 1

NUMBER OF HEADWAYS OBSERVED

Type of Headway	Passenger Cars			
	Left	Through	Right	Total
(a) Intersection A				
L	50	100	23	173
N	15	47	15	77
I	18	34	19	71
Total	83	181	57	321
(b) Intersection B				
L	21	75	17	113
N	7	87	10	104
I	14	70	23	107
Total	42	232	50	324
(c) Intersection C				
L	—	210	—	210
N	—	—	—	—
I	—	—	—	—
Total	—	210	—	210

TABLE 2

STATISTICAL DIFFERENCE OF MEANS OF HEADWAYS[1]

Description	Normal Test	"t" Test
(a) Intersection A		
Through N vs through L	Sig.	Sig.
Through N vs through I	Sig.	Sig.
Through L vs through I	Sig.	Sig.
Through L vs left-turning L	Non-sig.	Non-sig.
(b) Intersection B		
Through I vs through N	Non-sig.	Non-sig.
Through L vs through N	Sig.	Sig.
Through L vs through I	Sig.	Sig.
Through L maj. app. vs through L min. app.	Non-sig.	Non-sig.
(c) Combined Intersections A and B		
Through L vs rt-turn L	Sig.	Sig.
Through L vs lt-turn L	Non-sig.	Non-sig.
(d) Intersection A vs Intersection B		
Through N	Non-sig.	Non-sig.
Through I	Non-sig.	Non-sig.
Through L	Sig.	Sig.
Left-turn L	Non-sig.	Non-sig.
(d) Intersection C vs Intersection A and B		
Through L Int. C vs through L Int. A	Sig.	Sig.
Through L Int. C vs through L Int. B	Sig.	Sig.

[1] Passenger cars only.

fore the time taken by two successive vehicles on a loaded approach to proceed through the intersection, without any vehicle from the cross-street moving in between them, but only causing interference and hesitation. The 50-ft distance was selected as being representative of a critical lag of 3 to 4 sec, as found at four-way stops. A detailed analysis of acceptance and rejection of gaps at four-way stops was made by Cooper (2).

A total of 321 passenger car headways was recorded for Intersection A, 324 for Intesection B, and 210 for the lane studied at Intersection C. Table 1 gives the number of headways for each of the several items studied.

The size of the commercial vehicle sample was negligible. It is to be noted that the sample obtained from left- and right-turning vehicles is rather limited. In many instances, the data are very consistent, with a small standard deviation. In other cases, the data were combined for both Intersections A and B, and tested jointly.

Using a 95 percent confidence level, two types of tests were performed on various groups of data: (a) The normal distribution test (two-sided); and (b) The student's "t" distribution test (two-sided). The tests give the statistical significance of the difference of the means of two sets of data. The "t" test is preferable to the normal test for small samples, because it does not require the population values. The results (Table 2) indicate that left turns have no effect on the headway of departure, whereas right turns do. Except for the L headways of through passenger cars, computed on a total intersection basis, the split evidently has no effect. It must be agreed that this significant difference may also be influenced by location, sight distance, geometric configuration, etc., of the intersection. It so happens that Intersection B, which has relatively restricted sight distances when compared to Intersection A, gave shorter headways for through passenger cars. The three different types of headways (L, N, I) produced significant differences except in one case (see Table 2b). One of the important results obtained was the significantly longer headways needed to cross a four-lane vs a two-lane cross-street (Table 2e).

These analyses form a necessary basis from which one can estimate the effects of the various factors affecting the traffic behavior at intersections. These factors are considered quantitatively hereafter, and their influence on capacities is derived.

PRESENTATION AND DISCUSSION OF RESULTS

Table 3 gives the headways of departure of passenger cars for the conditions shown, as well as the standard deviation of each sample group. Figure 6 compares L-type mean headways for Intersections A and B; Figure 7 compares through mean headways for these intersections.

Factors Affecting Headway

Split. —Table 2d shows that the L type of headway of departure for through passenger cars is significantly different for Intersection A, with a split of 51.5/48.5, and Intersection B, with a split of 64.0/36.0 (Fig. 8). Because this type of headway (defined on loaded conditions at all times) is of interest in the calculation of basic capacity, the foregoing difference must be taken into account. Whether the difference is totally or partially due to the split, or to a certain unknown factor, cannot presently be determined and additonal research is needed.

If the split is assumed to be the most influential factor, it is possible to derive an equation of the headway as a function of the split. Inasmuch as only two different splits are available for study, the equation is the straight line:

$$H = 10.15 - 5S \qquad (2)$$

in which H is the average headway of departure for through passenger vehicles, for loaded condition; and S is the ratio of volume on major street to volume of total intersection. As S (split) increases, H (headway) decreases, and a larger volume of vehicles can be handled on the major approach. Although the capacity of the high-volume street increases, however, the minor-volume approach capacity decreases substantially, in order to satisfy the S (split) requirements. This obviously affects the capacity of the total intersection, for both basic and possible capacities.

Left Turns. —Under loaded conditions, left-turning passenger vehicles did not, as might have been expected, take a significantly longer time than through vehicles to proceed through the intersection (see Table 2c). This is undoubtedly due to the fact that many left turns are made simul-

TABLE 3

HEADWAYS OF PASSENGER CARS ENTERING A FOUR-WAY STOP INTERSECTION[1]

Intersection	Movement	Type of Headway	Mean (sec.)	Std. Dev. (sec.)
A	Through[1]	N	3.81	1.61
	Through[1]	I	4.73	1.86
	Through[1]	L	7.58	2.09
	Left[1]	L	7.40	2.22
	Right[1]	L	5.40	2.05
B	Through[2]	L	6.90	1.60
	Through[3]	L	7.04	1.18
	Through	N	4.18	1.36
	Through	I	4.28	1.62
	Through	L	6.96	1.51
	Left	L	7.57	2.12
	Right	L	6.38	2.06
A and B	Through	L	7.32	1.89
	Left	L	7.45	2.19
C	Through	L	8.08	1.03

[1] Averages for all four approaches.
[2] High-volume street.
[3] Low-volume street.

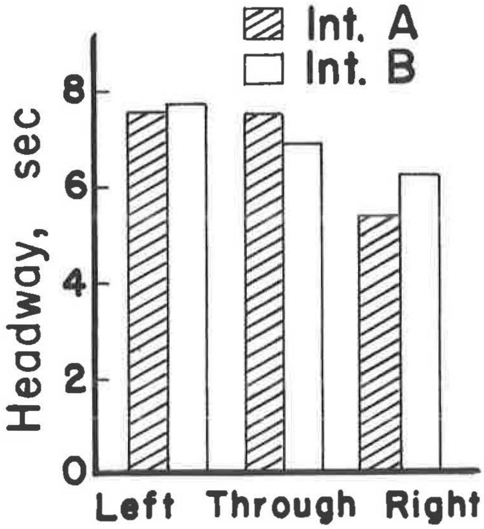

Figure 6. Comparative L-type mean headways, Intersections A and B.

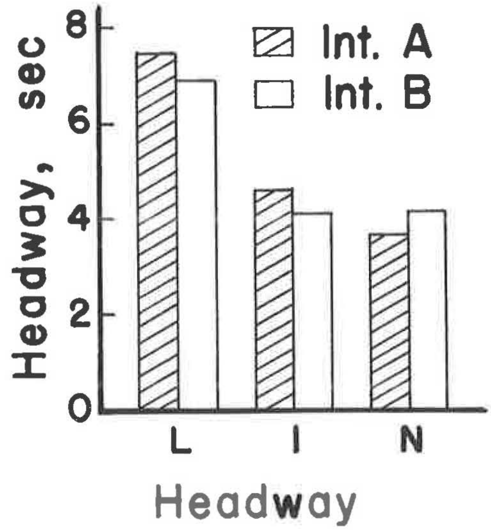

Figure 7. Comparative through mean headways, Intersections A and B.

taneously with right turns from the cross-street. These movements are not in conflict, and a high percentage of them were observed to be made simultaneously. It must be concluded that left turns have a negligible effect on the average headway, and, therefore, on the capacity of the four-way stop intersection with appreciable right-turning volumes.

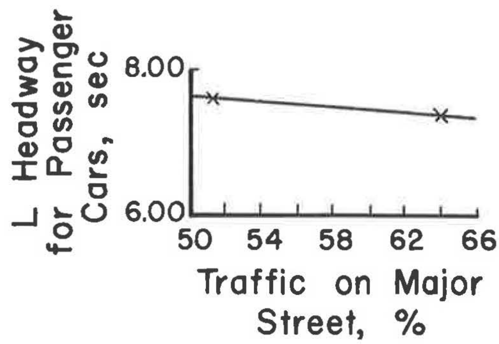

Figure 8. Variation of headway with split, Intersections A and B.

Right Turns.—It was observed that right-turning vehicles have significantly lower headways and, consequently, contribute to an increase in the capacity. This is not in accord with the Highway Capacity Manual (1), which states that, for traffic signal controls, capacity is decreased 0.5 percent for each 1 percent that right turns are of the total approach volume. This difference is easily understood if one examines the peculiar operation of the right turn at the four-way stop. A right-turning vehicle is not in conflict with either the right- or left-turning vehicle movements from the cross-street, and, furthermore, does not interfere with the through vehicles coming from the right-hand approach. During most of the time, right turns were performed simultaneously with other movements within the intersection area. The result is a smaller average headway. The fact also must not be overlooked that pedestrians, who mostly interfere with right turns, were totally absent from the intersections studied.

The mean headways for right turns and through vehicles, under loaded conditions, are, respectively, 5.82 and 7.32 sec, or a difference of 7.32 - 5.82/7.32 = 20.5 percent. In other words, for each 1 percent that right turns are of the through movement, the capacity is increased by 0.2 percent. Further research is required to verify the validity of this finding.

Commercial Vehicles.—As previously mentioned, data on commercial vehicles were negligible and no specific conclusions can be made as to their effect at four-way stops. However, it appears reasonable to assume their effect to be analogous to that observed at traffic signals. As reported in the Manual (1) capacity is reduced by 1 percent for each 1 percent that commercial vehicles are of the total traffic.

Pedestrians, Parking, Type of Urban Area. —The effects of pedestrians, parking, and type of urban area were not evaluated in this study.

Capacities

Basic Capacity. —As defined, the basic capacity is the maximum number of passenger vehicles that can be handled in one hour, under the most ideal conditions. This requires the intersection to be fully loaded, with queues of waiting vehicles on its approaches.

Two-Lane Street vs Two-Lane Street. —Referring to the specific operations of four-way stops, maximum volumes are handled when vehicles on opposite approaches accelerate simultaneously, alternating with the cross-street vehicles. Obviously, this is highly idealistic, and observed vehicle performance at four-way stops differs from it to a certain extent. The ideal performance is shown in Figure 9, with the two vehicles numbered 1 accelerating simultaneously across the intersectional area while the two vehicles numbered 3 move forward to the first position. Successively, vehicles 2, and then vehicles 3, 4, 5, etc., pass through the intersection. This procedure results in an equal volume of traffic being handled on both streets, and a split of 50/50. Under such conditions, the average headway on any approach can be calculated from Eq. 2; with $S = 0.50$, $H = 7.65$ sec. The basic capacity of the total intersection then becomes 3,600 sec/hr/7.65 sec/veh × 4 approaches = 1,885, or approximately 1.900 passenger cars per hour.

It is interesting to note that if vehicles would be entering the intersection from two opposite approaches only, with a clear cross-street at all times for one hour (split = 100/0), headways drop to 4.05 sec and the capacity for the whole intersection becomes 1,780, or approximately 1,800 passenger cars per hour.

This is 100 P.C./hr less than for the 50/50 split, as previously computed. This points out that the maximum number of passenger cars handled by two-way stops at the entrance to an arterial street is 900 per one-lane approach per hour. In order to do so, there must be no traffic on the arterial, and a continuous reservoir of vehicles at the stop sign. Any other actual conditions decrease the capacity.

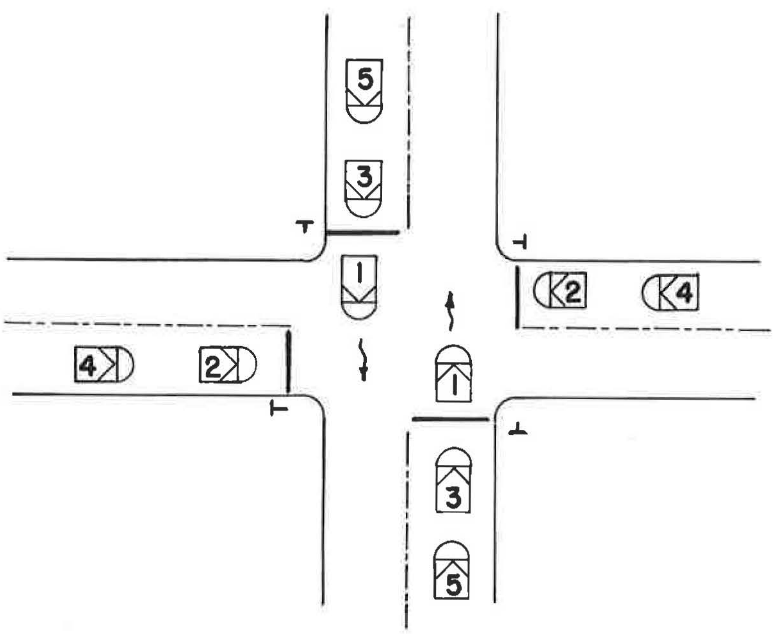

Figure 9. Ideal vehicle performance at 4-way stops.

It was observed that a certain number of vehicles at four-way stops do not always comply with the normal in-turn movement procedure for loaded conditions. Some bold drivers have a tendency to follow the vehicle immediately ahead of them through the intersection, thus proceeding across from the second position instead of the first. This behavior obviously causes a longer headway for the cross-street vehicles, because these must allow time for two vehicles to accelerate through instead of one. However, a shorter gap—which balances the one before—is produced by the bold driver, and the total capacity of the intersection is unaffected.

The previously computed basic capacity of 1,900 P.C./hr applies for a 50/50 split, which normally results when the two intersecting streets are loaded. The capacity for a different split can be calculated from (Table 4).

$$\text{Tot. int. basic cap.} = \text{vol. on loaded street} + \text{vol. on other street}$$

$$= \left(\frac{3,600}{10.15 - 5S}\right) \times 2 + \left(\frac{3,600}{10.15 - 5S}\right) \times 2 \times \left(\frac{1 - S}{S}\right)$$

$$= \frac{7,200}{(10.15 - 5S)S} \qquad (3)$$

Four-Lane Street vs Four-Lane Street.—As already pointed out in Table 2e, it took significantly longer for vehicles to cross the four-lane cross-street at Intersection C than the two-lane cross-streets of Intersections A or B. This is reflected in a longer headway of departure (8.08 sec). Assuming, on a two-lane approach basis, the ideal traffic behavior depicted in Figure 9 (i.e., simultaneous movement of vehicles entering from opposite approaches), the maximum capacity of this type of four-way stop intersection (split = 50/50) is 3,600/8.08 x 8 moving lanes = 3,570 P.C./hr, or approximately 3,600 P.C/hr.

Two-Lane Street vs Four-Lane Street.—Applying the foregoing assumptions, the maximum volume that can be handled by the intersection of a two-lane street with a four-lane street is (3,600/8.08 x 2) + (3,600/7.65 x 4) = 2,790, or approximately 2,800 P.C/hr.

The foregoing volumes are extremely high, and could only be attained under the most ideal conditions of roadway and traffic. The delays experienced by the waiting vehicles would be intolerably great. Under the best of prevailing conditions, and taking into account the effect of inept drivers and a variety of other factors, these theoretical capacities are impossible for most, if not all, four-way stop intersections.

Possible Capacity.—

Two-Lane Street vs Two-Lane Street.—The possible capacity of a four-way stop is equal to its basic capacity, adjusted for the specific conditions of the intersection. It is suggested that for two-lane vs two-lane streets the basic capacity values of Table 4 be used.

Adjustment factors for use with these basic capacities are then as follows:

1. Left turns: No adjustment.
2. Right turns: Increase capacity by 0.2 percent for each 1 percent that right turns are of the total traffic.
3. Interference factor: Because the values derived were from observations in outlying areas where no interference was encountered, it is suggested that a reduction factor be used for intersections in intermediate and downtown areas. More research is needed to determine the value of this factor. For the present, a value of 0.9 is suggested.

TABLE 4

BASIC CAPACITY[1] OF INTERSECTIONS FOR VARIOUS TRAFFIC SPLITS

Split	Basic Capacity (vph)
50/50	1,900
55/45	1,800
60/40	1,700
65/35	1,600
70/30	1,550

[1] Passenger cars per hour.

4. Commercial vehicles: Reduce capacity by 1 percent for each 1 percent that commercial values are of total traffic.

Four-Lane Street vs Four-Lane Street. —An analysis of Intersection C revealed that 702. percent of the time two vehicles waiting abreast on a two-lane approach moved simultaneously across the intersection. Of 336 cases observed, 236 pairs of vehicles accelerated at the same time. This obviously had an effect on the capacity. Taking the conservative value of $^2/_3$ for simultaneous movements, the possible capacity of the total intersection is $(3,600/8.08 \times 8 \times {^2/_3}) + (3,600/8.08 \times 4 \times {^1/_3}) =$ 2,970 P.C./hr, or approximately 3,000 P.C./hr. The adjustment factors are the same as for the previous case.

Two-Lane Street vs Four-Lane Street. —If one considers that two-thirds of the time two vehicles abreast on the four-lane street move simultaneously, and one-third of the time only one does, the possible capacity for this type intersection is $(3,600/8.08 \times 2) + (3,600/7.65 \times 4 \times {^2/_3}) + (3,600/7.65 \times 2 \times {^1/_3}) = 2,460$, or approximately 2,500 P.C./hr. The adjustment factors are the same as for the previous case.

Example of Application. —Intersection is two-lane vs two-lane; right turns = 10 percent; left turns = 10 percent; commercial vehicles = 5 percent; interference factor = 0.9; split = 60/40.

Required: To find possible capacity if present split is maintained. Using Table 10, possible capacity = $1,700 \times 0.9 \times 0.95 \times 1.02 = 1,700 \times 0.873 = 1,500$ vph.

Required: To find ultimate possible capacity (split = 50/50). Possible capacity = $1,900 \times 0.873 = 1,660$ vph.

Practical Capacity. —The Highway Capacity Manual (1) definition of practical capacity is based on the fact that most drivers are able to clear the intesection without waiting for more than one complete cycle. If the normal length of a cycle is assumed to be, say, 50 sec, it becomes possible to apply this definition to four-way stop intersections. Quoting the Manual: "With the normal short-time variation in flow, practical intersection capacities have been found to be approximately 80 percent of the possible capacities." Applying this factor, combined with the interference factor, practical capacities (before adjustments) are as follows:

Two-Lane Street vs Two-Lane Street. —Table 5 gives the practical capacities, in passenger cars per hour, for two-lane vs two-lane streets. The adjustment factors, except for interference factor, are the same as for possible capacity.

Four-Lane Street vs Four-Lane Street. —Practical capacity = $3,000 \times 0.9 \times 0.8 = 2,160$, or approximately 2,200 P.C./hr. The adjustment factors are the same as for the previous case.

Two-Lane Street vs Four-Lane Street. —Practical capacity = $2,500 \times 0.72 =$ 1,800 P.C./hr. The adjustment factors are the same as for the previous two cases.

POISSON DISTRIBUTION APPLIED TO DELAYS AND PRACTICAL CAPACITY

Because most driving inconveniences result from unnecessary waiting at intersections, delay is undoubtedly the best-suited single criterion on which to judge capacity. The Manual (1) definition of practical capacity is based on delay: 50 percent of vehicles or less waiting for a cycle length, or approximately 50 to 60 sec. If the rate of discharge of one lane through a four-way stop intersection is known, it is possible, assuming random arrival of vehicles and a Poisson distribution, to compute the volume of traffic

TABLE 5

PRACTICAL PASSENGER CAR CAPACITIES ON TWO-LANE vs TWO-LANE STREETS

Split	Practical Capacity (vph)
50/50	1,370
55/45	1,300
60/40	1,230
65/35	1,150
70/30	1,100

that will cause a certain percentage of drivers to wait for a certain selected time. Figure 10 shows approach volume as a function of percent of cleared periods, for 20-, 30-, 40-, and 50-sec periods. These periods are somewhat comparable to cycles; likewise, a period fails whenever more vehicles arrive at the intersection than can be discharged through. The average rate of vehicle discharge is taken as 6.00 sec, based on the assumption that 50 percent of the time, for practical capacity conditions, vehicles would enter the intersection alternately with the vehicles of the cross-street (L headway = 7.65 sec), and 50 percent of the time they would do so under no interference from the cross-street (N headway = 4.05 sec). Taking the average for the foregoing, $H = (7.65 + 4.05)/2 = 5.85$ sec, or a conservative value of 6.00 sec.

If, for instance, a 20-sec period is considered, whenever four or more vehicles arrive during that time, there is a failure, because on the average four vehicles require 24 sec to proceed through the intersection. For periods of 30, 40, and 50 sec, 5, 7, or 9 or more vehicles arriving during the respective periods produce a failure. From Poisson distribution, the probability of x vehicles arriving during time t is

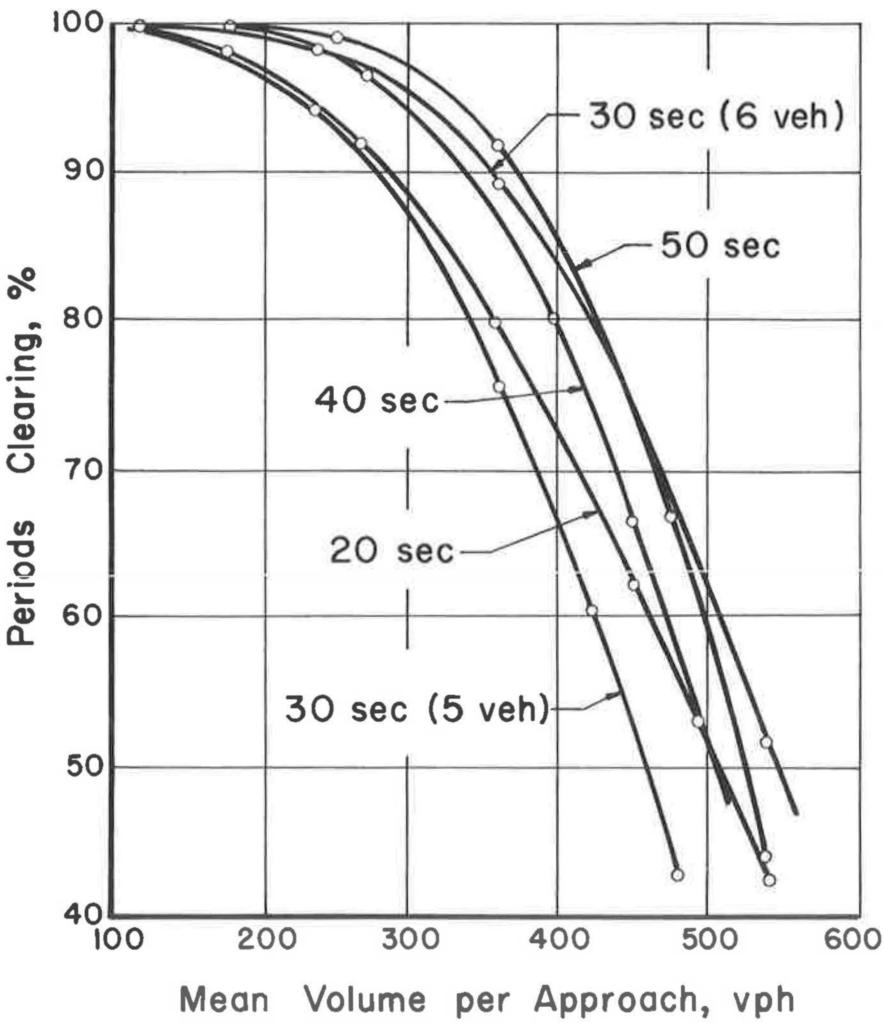

Figure 10. Relationship of approach volume to clearing period.

$$P(x) = \frac{e^{-m}\, m^x}{x!} \qquad (4)$$

in which

x = number of vehicles arriving during t sec;
m = mean number of vehicles arriving during t sec (=Vt/3, 600); and
V = vehicles per hour on the approach.

Inasmuch as $\sum^{\infty} P(x) = 1$, the probability of x or more vehicles arriving during time t is

$$P(x \text{ or more}) = 1 - P(\text{less than } x) = 1 - \sum_{0}^{x} \frac{e^{-m}\, m^x}{x!} \qquad (5)$$

For instance, if four or more vehicles fail a 20-sec period and a volume of 360 vph is assumed, m = 2 (or an average of 2 veh per 20-sec period) and P (4 or more) = 0. 143 or 14. 3 percent of failures. If the volume is 180 veh/hour, P (4 or more) = 0. 018 or 1. 8 percent of failures. It must be noted that a period failure affects the chances of the following period. Two periods (20 sec each) in a row will fail, for instance, if four vehicles arrive during the first and three during the second, or five during the first and two during the second or six during the first and one during the second. Summing up all these probabilities gives the probability of failure for any assumed volume.

The knee on each of the curves of Figure 10 is somewhere between 90 and 95 percent of cleared periods. A higher percentage of cleared periods requires an extremely low and unpractical volume on the approach, whereas a lower precentage increases delays very fast for a negligible additional volume on that approach. It is suggested, therefore, that practical capacity be of such a magnitude that 90 to 95 percent of the periods succeed in clearing.

It is debatable whether the period should be 20, 30, 40, or 50 sec. Obviously, the period length should be that which the majority of drivers is ready to accept as the maximum waiting time. Comparing the traffic behavior of a four-way stop (in which a driver moves forward toward the intersection in a step-by-step procedure, before "fighting" his way through) with the quiet and "almost relaxing" waiting in front of a red light, the maximum accepted waiting time is definitely much shorter than a cycle length, probably in the vicinity of 25 to 30 sec. Table 6 gives the volumes on one approach for four periods and three clearing percentages.

TABLE 6

VOLUME ON A ONE-LANE APPROACH

Periods Clearing (%)	Volume (vph)			
	20-Sec Period	30-Sec Period	40-Sec Period	50-Sec Period
90	285	6:355 319 5:282	335	372
92.5	262	6:333 296 5:258	314	353
95	233	6:312 270 5:228	288	330

An average of all Table 6 values gives a volume of 305 P.C./hr per lane of approach. Taking 300 P.C./hr as the practical capacity of a one-lane approach, it is found that 90 percent of the periods clear if their length is 24 sec, 92.5 percent do if their length is 33 sec, and 95 percent for a length of 41 sec. According to this criterion, the practical capacity of two-lane vs two-lane four-way stop intersections would be 300 × 4 = 1,200 P.C./hr, before adjustments.

CONCLUSIONS AND RECOMMENDATIONS

Because the results of this study are based on relatively limited data, it is possible that certain unknown location factors may have biased the samples. Therefore, additional data should be taken on the issue of four-way stop intersection behavior. Many characteristics of traffic, including commercial vehicles, parking, pedestrians, type of urban area, etc., were not covered. Their effects on the capacity of the type of intersection control studied are still to be determined. The observed intersections were located in the Chicago area only, and data from other parts of the country are needed. Some unexpected results, like the effect of left- and right-turning vehicles and split, demand further research. Additional studies should ascertain the effect of location and split on the capacity by analyzing a number of different intersections having the same split. Covariance, as a tool of statistical analysis, should be considered.

Table 7, which summarizes the most important results of this study, is suggested as a trial capacity chart for four-way stop intersections. It may be useful as an upper-limit volume warrant.

TABLE 7

CAPACITY CHART FOR FOUR-WAY STOP INTERSECTIONS WITH 50/50 SPLIT

Capacity Type	Capacity[1] (vph)		
	2-Lane × 2-Lane	2-Lane × 4-Lane	4-Lane × 4-Lane
Basic	1,900	2,800	3,600
Possible	1,900	2,500	3,000
Practical	1,200	1,800	2,200

[1] Before adjustments.

ACKNOWLEDGMENTS

This paper is based on a thesis submitted to the Department of Civil Engineering, Northwestern University, in partial fulfillment of the requirements for the degree of Master of Science.

Sincere gratitude and appreciation are extended to Professor Donald S. Berry, whose patient guidance, advice and inspiration made this study possible. Thanks also are expressed to Assistant Professor Sidney Singer, who helped in the statistical analysis of the results, and to Dr. Paul Shuldiner, for his advice and review.

REFERENCES

1. "Highway Capacity Manual." Supt. of Documents, U.S. Govt. Printing Office, Washington, D.C. (1950).
2. Cooper, B.K., "A Supplementary Message at Four-Way Stop Intersections." Thesis, Purdue Univ. (1957).
3. "Manual on Uniform Traffic Control Devices for Streets and Highways." Supt. of Documents, U.S. Govt. Printing Office, Washington, D.C. (1961).
4. Brown, L.R., "The Traffic Signal vs the Full Stop at Outlying Intersections." Proc. Inst. Traffic Eng. (1932).
5. Erickson, E.L., "Traffic Performance at Urban Street Intersections." Traffic Quart., Eno Foundation (July 1945).
6. Greenshields, B.D., "Traffic Performance at Urban Street Intersections." Yale Bureau of Highway Traffic (1947).
7. Greenshields, B.D., and Weida, F.M., "Statistics with Application to Highway Traffic Analyses." Eno Foundation (1952).
8. Hall, E.M., "A Comparison of Delay to Vehicles Crossing Urban Intersections Four-Way Stop vs Semi-

Traffic Actuated Signal Control." Student Res. Rept. No. 4, Inst. of Transportation and Traffic Engineering, Univ. of California (1952).

9. Hanson, D.J., "Are There Too Many Four-Way Stops?" Traffic Eng. (Nov. 1957).
10. Harrison, H.H., "Four-Way Stops." Traffic Eng. (Feb. 1949).
11. Holmes, E.H., "The Effect of Control Methods on Traffic Flow." Pub. Roads (Feb. 1934).
12. Keneipp, J.M., "Efficiency of Four-Way Stop Control at Urban Intersections." Traffic Eng. (June 1951).
13. McEachern, C., "A Four-Way Stop-Sign System at Urban Intersections." Traffic Quart., Eno Foundation (April 1959).
14. Raff, M.S., "Space-Time Relationships at Stop Intersections." Proc. Inst. Traffic Eng. (1949).
15. Raff, M.S., "A Volume Warrant for Urban Stop Signs." Eno Foundation (1950).
16. Raff, M.S., "A New Study of Urban Stop Signs: A Volume Warrant." Traffic Quart., Eno Foundation (Jan. 1950).
17. Wilkie, L.G., "58,732 Motorists Checked at Stop Signs." Traffic Eng. (April 1954).

Appendix

DISTRIBUTION OF PASSENGER CAR OBSERVATIONS

Headway Cell Length (sec)	Through Car Headways (no.)							L-Type Headways (no.)			
	Intersection A			Intersection B			Inters. C	Inters. A		Inters. B	
	L Type	N Type	I Type	L Type	N Type	I Type	L Type	Left Turns	Right Turns	Left Turns	Right Turns
0.0 - 0.9	-	0	0	-	0	0	-	-	-	-	-
0.9 - 1.5	-	1	0	-	1	0	-	-	-	-	-
1.5 - 2.1	-	5	1	-	3	2	-	-	-	-	-
2.1 - 2.7	-	9	4	-	8	11	-	0	0	0	0
2.7 - 3.3	-	6	3	0	8	10	-	0	2	0	3
3.3 - 3.9	0	6	4	2	18	8	0	0	4	0	0
3.9 - 4.5	4	3	4	2	15	8	0	4	4	1	0
4.5 - 5.1	3	7	6	3	13	11	8	3	3	3	3
5.1 - 5.7	8	5	3	7	11	8	2	6	1	1	4
5.7 - 6.3	9	2	3	12	7	4	4	7	3	5	1
6.3 - 6.9	18	1	2	10	0	5	10	3	2	3	2
6.9 - 7.5	15	1	2	12	1	1	17	3	1	3	1
7.5 - 8.1	10	0	0	12	2	0	67	7	1	0	2
8.1 - 8.7	10	1	0	7	0	0	53	3	0	1	0
8.7 - 9.3	11	0	1	4	-	2	32	5	0	0	0
9.3 - 9.9	3	-	1	2	-	0	17	2	0	1	0
9.9 - 10.5	1	-	0	1	-	-	0	2	1	0	0
10.5 - 11.1	1	-	-	0	-	-	-	2	1	2	0
11.1 - 11.7	1	-	-	1	-	-	-	0	-	1	1
11.7 - 12.3	1	-	-	0	-	-	-	2	-	0	0
12.3 - 12.9	4	-	-	-	-	-	-	1	-	-	-
12.9 - 13.5	0	-	-	-	-	-	-	0	-	-	-
13.5 - 14.1	0	-	-	-	-	-	-	-	-	-	-
14.1 - 14.7	0	-	-	-	-	-	-	-	-	-	-
14.7 - 15.3	2	-	-	-	-	-	-	-	-	-	-
15.3 - 15.9	0	-	-	-	-	-	-	-	-	-	-