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Effects of Traffic Control On Street Capacity

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Effects of Traffic Control On Street Capacity

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Capacities of Narrow Streets with Manual Control And Signal Control

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American Institute of Planners

● THIS paper reports on the capacities of narrow streets at Fort Belvoir, Virginia, as determined by field survey of capacities at intersections.

Capacities of approach lanes of intersections are presented as percentage overloads above practical capacity under local conditions, the latter being calculated according to Highway Research Board formulas published in the Highway Capacity Manual.

The general significance of the paper lies in the fact that it presents data bearing upon possible extension of the curves in Figure 24 of the Highway Capacity Manual entitled, "Average Reported Intersection Capacities for Two-Way Streets."

BACKGROUND

In recent years the Department of Defense has been stimulating the preparation of master plans for future development of Army, Navy and Air Force permanent installations. The master plan work has included surveys and the preparation of maps recording existing conditions. Information on existing conditions is taken as a starting point for preparation of a master plan.

Until recently the survey and recording of existing conditions had not extended into the field of comprehensive traffic surveys. One of the early ventures in this direction was the letting of a special contract for a comprehensive traffic survey[1] at Fort Belvoir, Virginia. The results of the traffic survey were to be used, along with other portions of the master plan work, in developing a master plan of streets and roads for the fort. The contract referred to was let by the Corps of Engineers, U. S. Army, Office of the District Engineer, Washington District, Washington, D. C., to the firm of Groll-Beach and Associates, Architects and Planning Engineers, Washington, D. C. Field work was carried on in July and August of 1953, and the report published under the title, "Vehicular Traffic Survey and Master Plan of Streets and Roads, Fort Belvoir, Virginia, September 1953."

EXTRAPOLATION OF FIGURE 24, HCM

In order to apply Highway Research Board formulas to the narrow streets of Fort Belvoir, it was necessary first to extrapolate the appropriate curves on the graph of average reported intersection capacities published as Figure 24 of the Highway Capacity Manual. In this figure as published the curves do not extend to streets of narrower width than thirty feet, while most of the streets in Fort Belvoir are between eighteen and twenty four feet wide.

The curves selected for extrapolation were the "intermediate with parking prohibited" and the "outlying or rural." Belvoir streets are generally too narrow to permit on-street parking; there is not enough pedestrian traffic nor enough closely developed building frontage to justify a downtown classification; and in many cases the absence of curb and gutter and the open character of adjoining land make the outlying or rural classification quite accurate.

Advice was sought from several experts as to the character of an appropriate extrapolation (See Fig. 1). Using a 20 ft. wide street as a check point, Mr. O. K. Normann, co-author of the Highway Capacity Manual, suggested a figure of 400 vehicles per hour of green, Mr. Prisk of the Bureau of Public Roads 450, and Mr. Henry Evans, editor of the Traffic Engineering Handbook, 540. An intermediate figure of 500 was adopted and the two curves were extended in a straight line to that point.

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[1] Street and intersection traffic volumes and capacities, speed-and-delay, origin-and-destination, accident, parking, signs and markings, signal operation.

CALCULATION OF PRACTICAL CAPACITY FOR LOCAL CONDITIONS

Practical capacities under local conditions were calculated according to the formulas of the Highway Capacity Manual, using the extrapolated curve reduced ten percent to practical capacity, and using field data on widths of approach lanes, percents of commercial vehicles, right turns and left turns, presence of parking and bus stops, and minutes of green time. Details of these calculations are presented in Table 7 at the end of the paper.

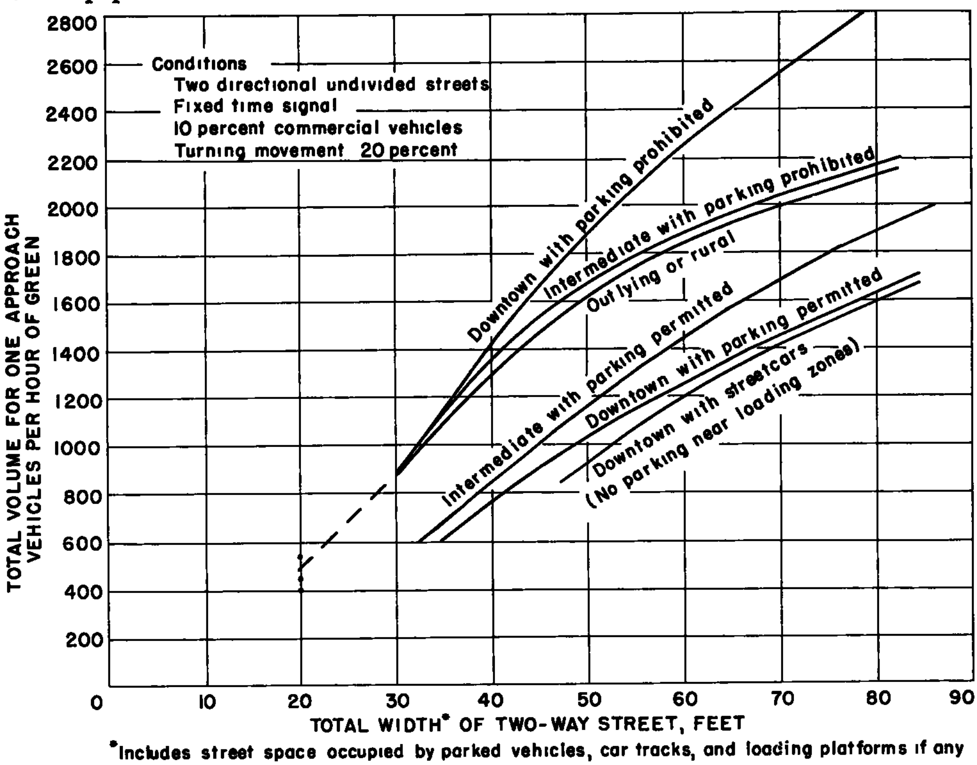


Figure 1. Average reported intersection capacities for two-way streets by type of area and parking regulation. (See text for description of annotations).

OVERLOADS

When measured flows were compared with calculated practical capacities, and the excess designated overload, the following percentage overloads were found to exist during the peak hour:

TABLE 1

PEAK HOUR OVERLOADS ON APPROACHES TO INTERSECTIONS, IN ORDER OF SIZE

No. of approaches	Overload (percent)
1	100
3	80
1	70
1	60
3	50
3	30
2	20
1	10

These data are presented graphically in Figure 2, a bar chart in which each horizontal bar represents one overloaded approach to an intersection. The outlined portion of the bar, on the left side of the figure, represents capacity in use, in each case this being 100 percent of practical capacity for local conditions, except for one factor. Only five of these approaches were controlled by fixed time signals. Practical capacities for the other ten were calculated as though they were controlled by fixed time signals. The solid portion of each bar, on the right side of the figure, represents

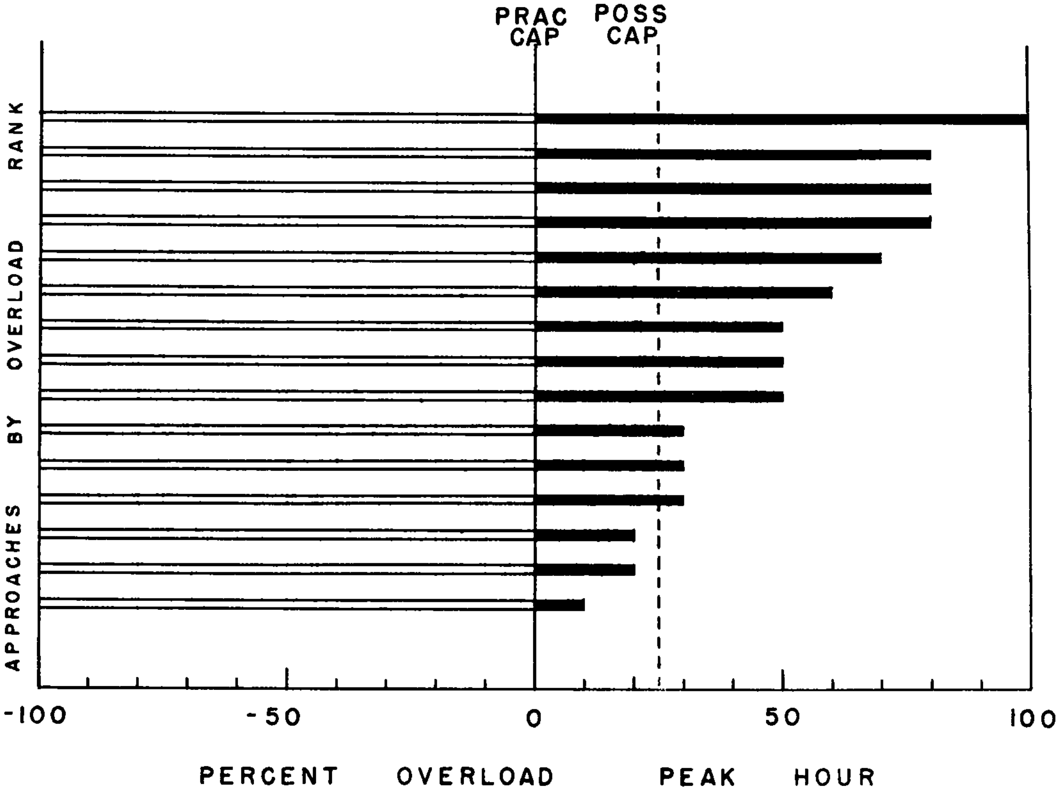


Figure 2. Peak hour overloads on approaches to intersections, in order of size.

actual traffic flow in excess of calculated practical capacity.

The vertical dashed line represents the limit of possible capacity under the formula that practical capacity is about eighty percent of possible capacity. The reciprocal of this relationship is that possible capacity is about 125 percent of practical capacity.

The bar chart indicates a fairly even distribution of overloads through the range from ten to 100 percent. As a check on this general impression, however, another bar chart showing frequency distribution of overloads by ten percent classes is given in Figure 3. This chart indicates that there is no pronounced concentration of overloads at any point in the general range of overloads.

When the peak hour overloads are segregated according to the type of control exercised at intersections, the following breakdown is obtained:

TABLE 2

PEAK HOUR OVERLOADS ON APPROACHES TO INTERSECTIONS, BY TYPE OF CONTROL

Type of control	No of approaches	Overload (percent)
Military police[1]	1	100
	1	80
	1	70
	3	30
Traffic actuated signal	1	80
	1	60
	1	50
Fixed time signal	1	80
	2	50
	1	20
	1	10
None	1	20

[1] Either by direct manual control or by pushbutton operation of signals

The same results are presented graphically in Figure 4.

It will be noted from these data that intersection approach capacity under conditions of overloading appears to vary directly with the degree to which traffic control devices can respond to excessive traffic demands.

Figure 5, which is Plate 16 from the final report, shows the manner in which peak hour overloads were presented in map form. In this figure north lies to the left, the Potomac River is to the right, the Belvoir peninsula is bounded by Dogue Creek at the top and Accotink Bay at the bottom. The

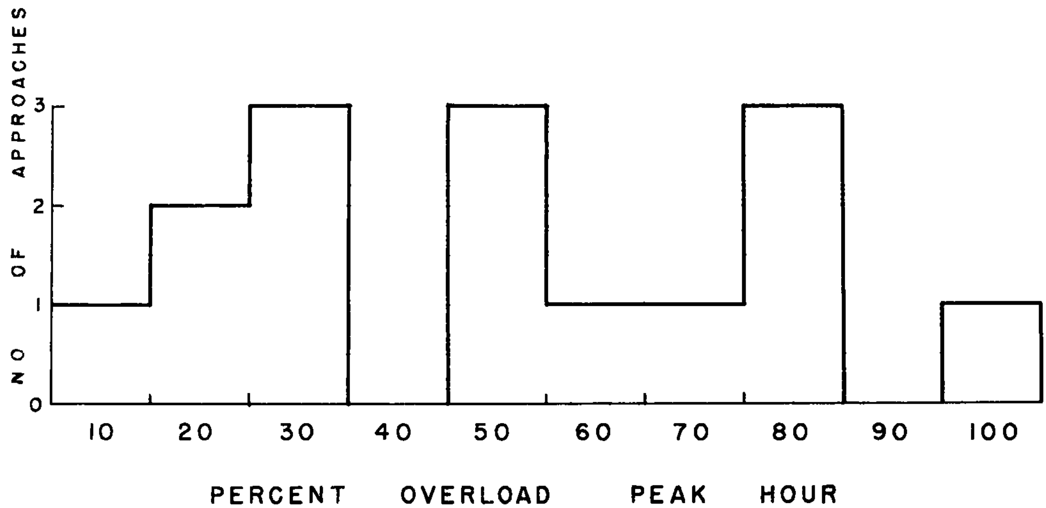


Figure 3. Frequency distribution of peak hour overloads on approaches to intersections, by size of overload.

Washington-Richmond highway, US 1, runs vertically through the figure at the left third point. The Main, or South Post lies to the right of US 1, the smaller North Post to the

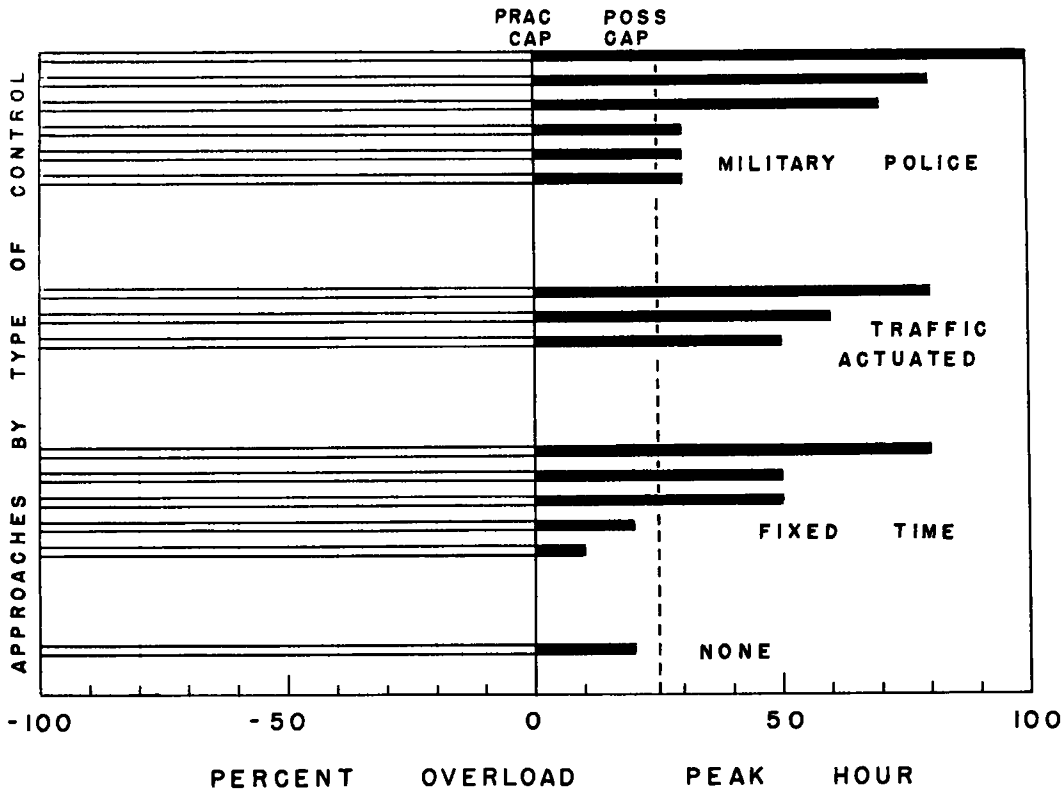


Figure 4. Peak hour overloads on approaches to intersections, by type of control.

left. The upper horizontal road is Belvoir Road, the main entrance to the fort. The lower horizontal road is Gunston Road which overpasses US 1 to connect North and South Posts.

Of the color annotations, white indicates the intersections which were studied; dark tone, capacity in use; light tone, capacity available but not used; and black, recorded flow above capacity, or overload. Colored bands for one approach to an intersection are carried back along the approach road to the approximate midpoint between the intersections studied. On some of the less important intersections, unused capacity is not shown.

The relative widths of the dark tone and black bands show the degree of overloading, while the actual width of the black band shows the volume of traffic passing through the intersection as overload.

The data presented thus far symbolize aggregate overloads during the peak hour without revealing variations in the degree of overloading during the hour. It is by no means certain that all these approaches were loaded on each cycle or go period during the peak hour. Specific field records were not kept on this point. Based upon general knowledge of traffic conditions at the fort, it is believed, however, that at least one approach was definitely not loaded on all cycles, and that five others were probably not loaded on all cycles. The peak hour analysis has been made primarily to permit correlation of these data with other peak hour figures such as those given in the Highway Capacity Manual.

TABLE 3

PEAK 15 MINUTE OVERLOADS ON APPROACHES TO INTERSECTIONS, IN ORDER OF SIZE

No. of approaches	Overload (percent)
1	260
1	210
1	200
1	190
1	170
1	160
1	140
1	130
1	110
3	100
1	90
1	80
1	70
3	60
1	50
1	40
3	30
6	20
2	10

For the purpose of calculating and designing the required enlargement of intersections at Fort Belvoir, a comparable analysis of peak 15 minute data was made.

Thirty-one approaches to intersections were found to be loaded above calculated practical capacity under local conditions. Overloads ranged from ten to 260 percent. Table 3 and Figure 6 show these overloads arranged according to rank, while Figure 7 shows the frequency distribution of overloads in ten percent bands. Generally speaking, the heaviest grouping of overloads comes in the zero to 100 percent band; there is a marked reduction in the 100 to 200 percent band; and only two items fall in the 200 to 300 percent band. When the peak 15 minute overloads are arranged according to type of traffic control at intersections, the same sequence is observed as with peak hour data, that the highest overloads are obtained under military police control, the next highest with traffic-actuated signals, and the lowest, aside from uncontrolled approaches, with fixed-time signals. Table 4 and Figure 9 show the details of arrangement by type of control. The greater relative size of the group of approaches under military police control is due to the fact that the post has only one traffic-actuated and two fixed-time signals. As more intersections become overloaded during thirty or forty-five minute rush periods, more military police teams are assigned to rush hour traffic control.

Figure 8 shows the peak 15 minute overloads in map form. It is readily apparent that percentage overloads are higher than during the peak hour and that more approaches are overloaded.

SIGNIFICANCE OF THE FINDINGS

Since the primary purpose of this paper is to present the findings rather than to an-

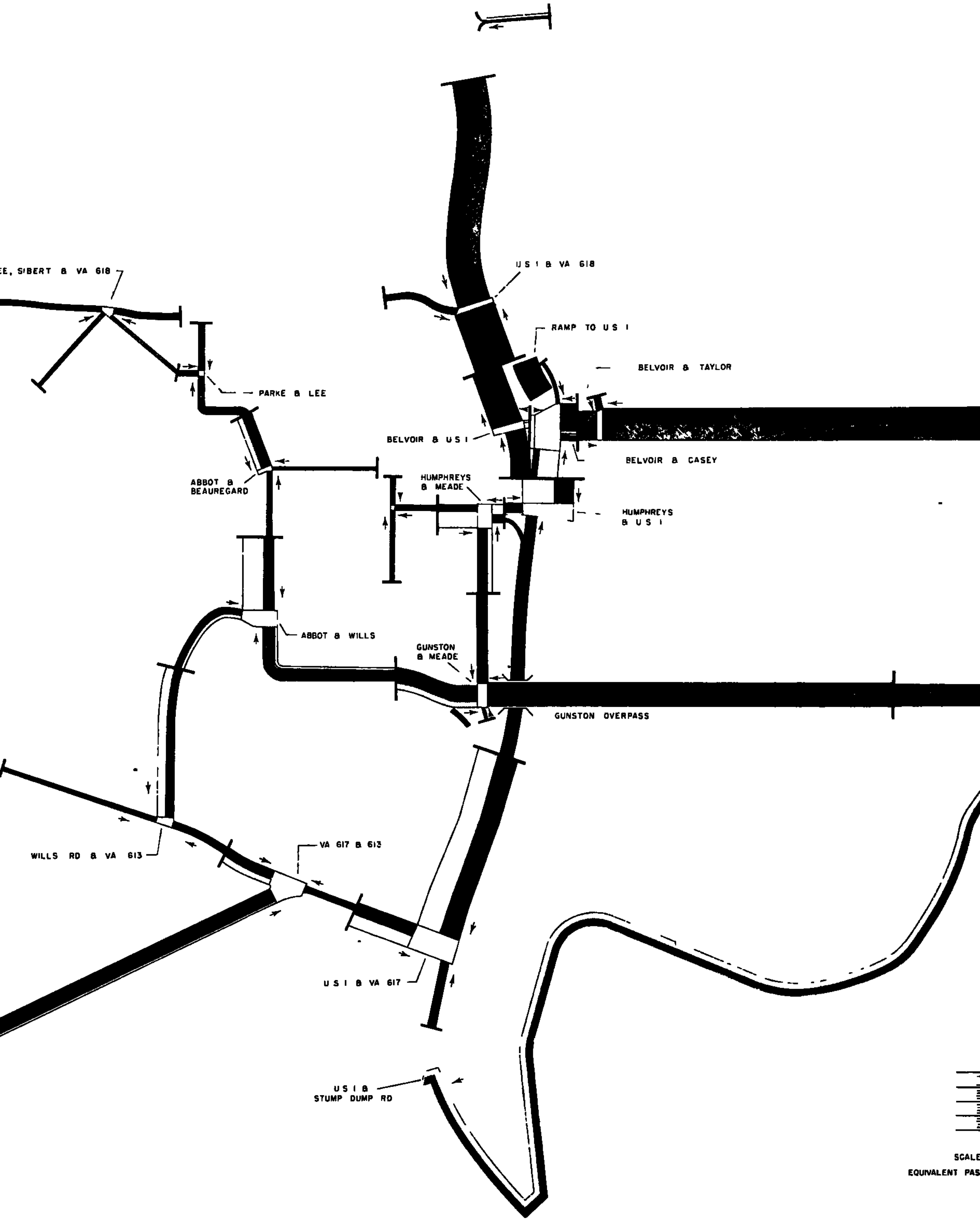
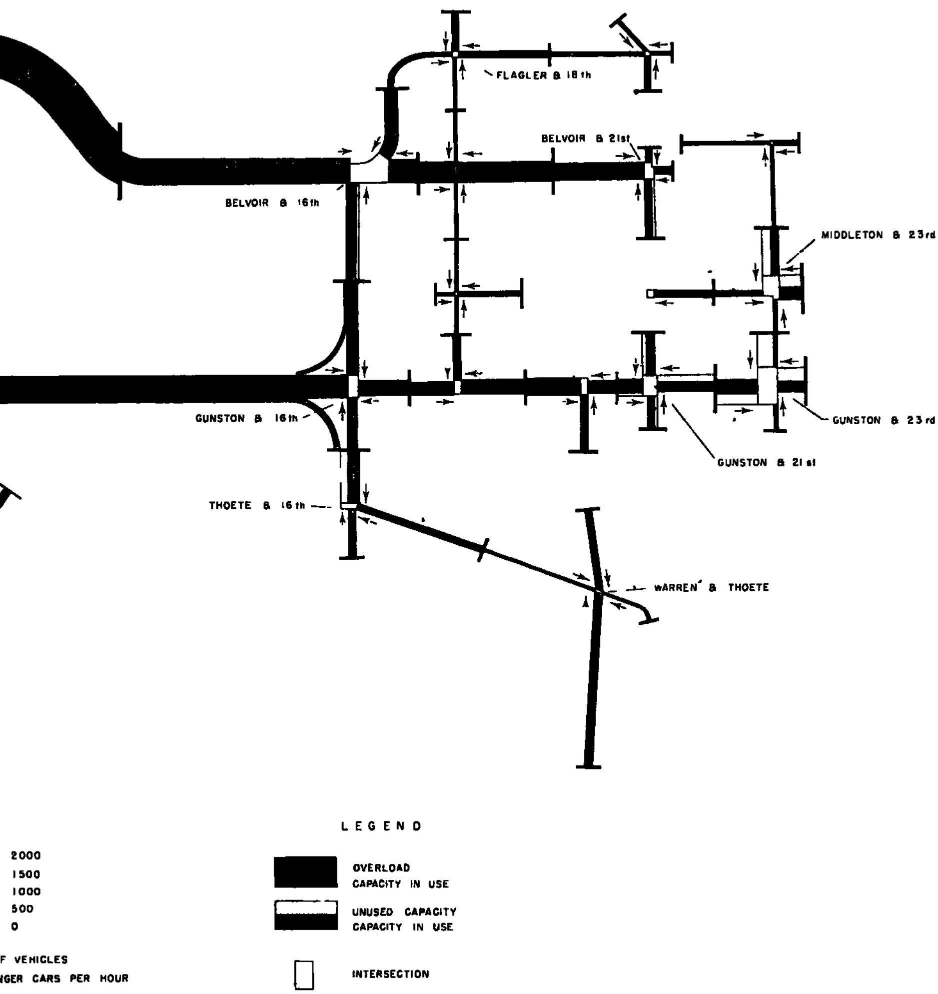


Figure 5. Peak-hour flows, capacities, overloads, and unused capacities on approach



lanes of intersections, Fort Belvoir, Virginia. Courtesy of Groll-Beach & Associates.

TABLE 4

PEAK 15 MINUTE OVERLOADS ON APPROACHES TO INTERSECTIONS, BY TYPE OF CONTROL

Type of control	No. of approaches	Overload (percent)
Military police	1	260
	1	210
	1	190
	1	140
	1	130
	1	110
	1	100
	1	90
	2	60
	1	50
	2	30
	3	20
	2	10
Traffic actuated signal	1	200
	1	160
	1	80
Fixed time signal	1	170
	2	100
	1	70
	1	60
	1	20
None	1	40
	1	30
	2	20

TABLE 5

SAMPLE SPEEDS ON APPROACHES TO INTERSECTIONS

Length of approach (ft.)	Elapsed time (min. and sec.)	Average speed (mph.)
1900	8:30	2.5
3700	13:02	3.2
3000	10:02	3.4
10000	19:27	5.8

TABLE 6

SAMPLE SEQUENCE OF STOP PERIODS FOR MINOR FLOW AT INTERSECTION (min. and sec.)

1:11
2:15
1:48
:58
2:40
:35
2:27
:20
1:40
2:25
:25
:25
1:18
1:56

alyze their significance, only a few comments will be made under this heading.

In addition to the overloads which were pushed through Fort Belvoir intersections during peak periods, there were frequently tremendous backlogs of vehicles unable to transit the intersections. Speed and delay runs on four intersection approaches yielded the results shown in Table 5. A speed and delay run through the main entrance of the post to a popular destination covered 2.9 miles in 23 minutes and 15 seconds for an average speed of 7.4 miles per hour.

At intersections under military police control, the overloads were generally accomplished by stretching the go period for the major flow to such extreme lengths that cross traffic was severely penalized. During a thirty minute period one minor traffic stream, though itself a rush hour home-to-work movement, was held by military police for the stop periods shown in Table 6.

Of course, such excessive single-cycle delays to cross traffic could not occur at intersections controlled by fixed-time signals, although lengthy back-ups did occur. The observed high rates of flow at such intersections must have been stimulated in part by close driver familiarity with road and traffic conditions, and in part by uniform driver motivation, e.g., a desire to get to work on time or to get home as quickly as possible.

After allowances have been made for continuously loaded approaches, highly unusual operating conditions, and rush hour driver characteristics, the question is raised, but by no means answered in this study, whether the curves in Figure 24 of the Highway Capacity Manual, if carried down to narrower street widths, should not tend to flatten out toward the horizontal as they reach the range of two-way streets 18 to 24 ft. wide.

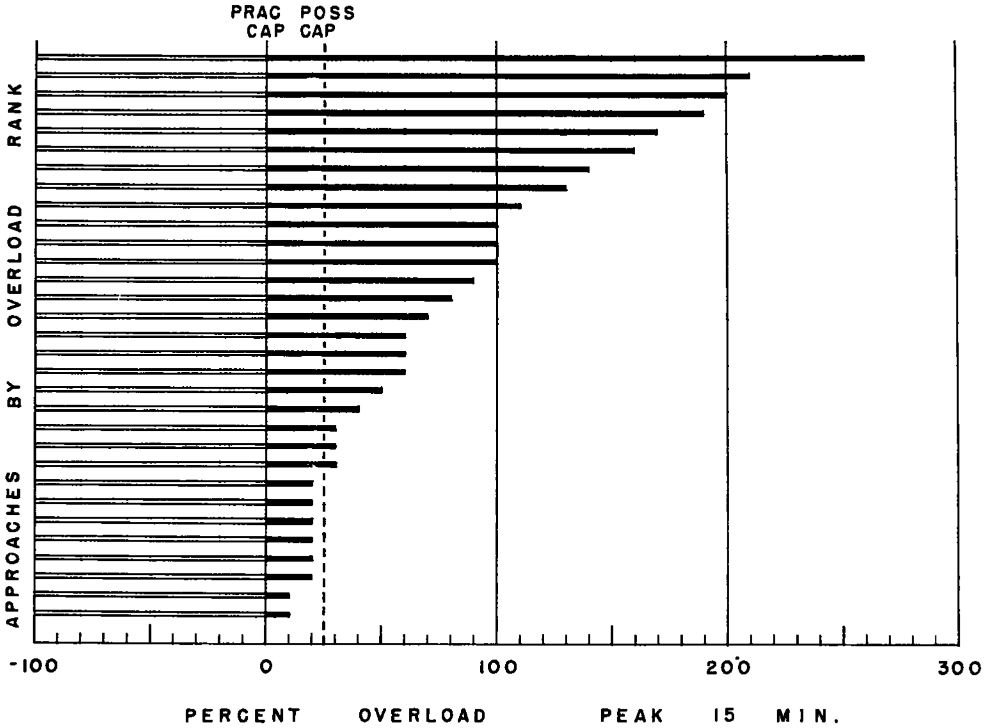


Figure 6. Peak 15 minute overloads on approaches to intersections, in order of size.

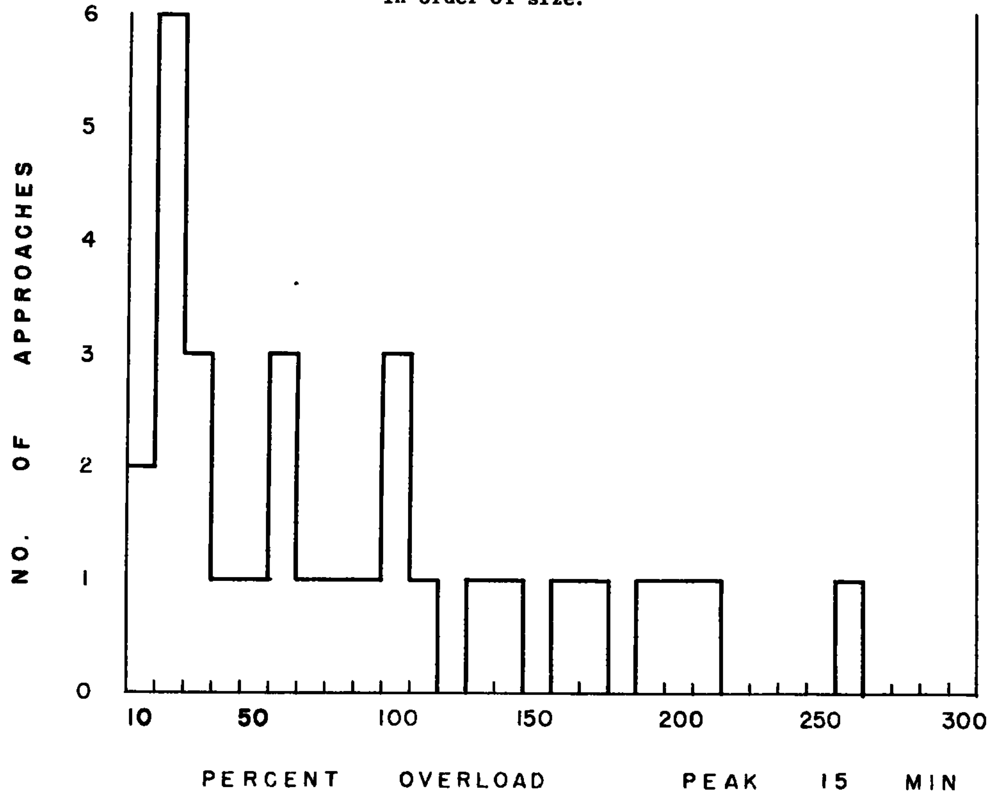


Figure 7. Frequency distribution of peak 15 minute overloads on approaches to intersections, by size of overload.

TABL

TRAFFIC STREAM

COMPUTAT

Intersection	Traffic Bound	Entering Intersection On	Width of Approach Lane Feet	Length of Peak Flow Period Minutes	Time of Peak Flow Hour of Day	Practical Capacity of Approach Lane Under Average Conditions Veh/hr	ADJUSTMENT: Commercial Vehicles (1) No	Commercial Vehicles (1) Adj %	Right Turn No	Right Turn % $\frac{(10)}{(23)}$	Right Turn Adj (2) %	No.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Belvoir Rd & 21st St	South	Belvoir Rd	12' - 9"	60	0715-0815	648	25	+10	103	23 0	-6 5	96
				15	0730-0745	162	11	+10	38	25 5	-7 7	25
Belvoir Rd & 21st St	East	21st St	11' - 0"	60	1630-1730	522	2	+10	2	1 0	+5 0	152
				15	1630-1645	130	1	+10	1	1.0	+5.0	58
Belvoir Rd & 16th St	South	Belvoir Rd	12' - 4"	60	0700-0800	618	23	+10	99	13 2	-1 6	27
				15	0745-0800	154	9	+10	35	14 4	-2 2	10
Belvoir Rd & 16th St	North	Belvoir Rd	12' - 0"	60	1630-1730	594	19	+10	0	0 0	+5 0	32
				15	1645-1700	149	3	+10	0	0 0	+5 0	13
Belvoir Rd & 16th St	East	16th St	15' - 4"	60	0730-0830	834	67	+10	102	43 7	-10 0	124
				15	0745-0800	218	20	+10	44	T	+5 0	40
Belvoir Rd & Harris Rd	West	Harris Rd	10' - 6"	60	1630-1730	486	7	+10	317	T	+5 0	NP
				15	1700-1715	121	0	+10	131	T	+5.0	NP
Belvoir Rd & Taylor Rd	North Thru lane only	Belvoir Rd	12' - 0"	60	1630-1730	594	28	+10	0	0	+5 0	
				15	1645-1700	149	5	+10	0	0	+5 0	
Belvoir Rd & Taylor Rd	South	Belvoir Rd	11' - 6"	60	0700-0800	558	27	+10	NR	NR	+5 0	49
				15	0730-0745	139	2	+10	NR	NR	+5 0	16
Belvoir Rd & Casey Rd	North R lane only	Belvoir Rd	12' - 9"	60	1630-1730	648	40	+10	81	8 4	+1 0	0
				15	1645-1700	162	6	-10	24	8 3	+1 0	0
Belvoir Rd & Casey Rd	East	Ramp (S)	21' - 0"	60	0700-0800	1310	17	+10	192	93 0	-10 0	0
				15	0730-0745	338	4	+10	71	100 0	-10 0	0
Belvoir Rd & Casey Rd	South	Belvoir Rd	20' - 6"	60	0700-0800	1295	23	+10	NP	NP	+5 0	5
				15	0730-0745	324	6	+10	NP	NP	+5.0	0
Belvoir Rd & U S No 1	North Lt only	Belvoir Rd	10' - 0"	60	1630-1730	1000 (4)	18	0	(4)	(4)	(4)	172
				15	1645-1700	250 (4)	5	0	(4)	(4)	(4)	50
Belvoir Rd & U S No 1	West Lt turn lane only	U S No 1 -	10' - 0"	60	0700-0800	1000 (4)	42	0	(4)	(4)	(4)	(4)
				15	0730-0745	250 (4)	12	0	(4)	(4)	(4)	(4)
Middleton Rd & 23rd St	North	Putman	9' - 10" 19' - 8"	60	1600-1700	621 (6)	3	+10	192	55 5	-10 0	0
				15	1630-1645	292	0	+10	162	55 6	-10 0	0
Middleton Rd. & 23rd St	West	23rd St	12' - 1"	60	0700-0800	600	4	+10	5	2 2	+ 4 0	191
				15	1730-1745	150	3	+10	0		+ 5.0	87
Gunston Rd & 23rd St	South	Gunston	12' - 6"	60	0700-0800	630	18	+10	15	3 8	+ 3 0	62
				15	0730-0745	158	7	+10	6	3 1	+ 3 5	31
Gunston Rd & 23rd St	North	Gunston	11' - 3"	60	1600-1700	540	9	+10	15	5 5	+ 2 0	
				15	1630-1645	135	4	+10	9	4 5	+ 3 0	
Gunston Rd & 23rd St	West	23rd St (S)	12' - 0"	60	1200-1300	594	4	+10	13	20 0	- 5 0	16
				15	1215-1230	148	2	+10	5	22 7	- 6 0	6
Gunston Rd & 21st St	South	Gunston	12' - 3"	60	0715-0815	612	29	+10	23	5.9	+ 2.0	88
				15	0730-0745	153	9	+10	5	2 7	+ 4 0	22
Gunston Rd & 21st St	North	Gunston	12' - 3"	60	1615-1715	612	11	+10	45	15 4	- 2 7	44
				15	1630-1645	153	7	+10	33	14 6	- 2 0	33
Gunston Rd & 21st St	West	21st St (S)	11' - 0"	60	1630-1730	522	5	+10	96	47 7	-10 0	8
				15	1700-1715	130	1	+10	63	61 7	-10 0	2
Gunston Rd & 21st St	East	21st St (S)	11' - 0"	60	0715-0815	522	9	+10	68	44 1	-10 0	23
				15	0730-0745	130	0	-10	48	69 5	-10 0	2
Gunston Rd & 16th St	South	Gunston	11' - 9"	60	0715-0815	576	86	-10	0	0 0	+ 5 0	30
				15	0745-0800	144	25	-10	0	0 0	+ 5 0	9
Gunston Rd & 16th St	North	Gunston	11 - 11"	60	1630-1730	588	45	+10	54	13 0	+ 1 5	25
				15	1645-1700	147	14	+10	21	7 9	+ 1 0	12
Gunston Rd & 16th St	West	16th St	12' - 3"	60	0715-0815	612	88	-10	0	0 0	+ 5 0	57
				15	0745-0800	153	25	-10	0	0 8	+ 5 0	23
Gunston Rd & 16th St	East	16th St	11' - 9"	60	1615-1715	576	40	-10	19	¯ 1	+ 1 0	113
				15	1630-1645	144	12	+10	7	7.7	+ 1.0	36
Gunston Rd & Meade Rd	North	Gunston	15' - 0"	60	1630-1730	810	67	-10	327	39 9	-10 0	3
				15	1700-1715	202	11	+10	93	39 2	-10 0	0
Gunston Rd & Meade Rd	South	Gunston	12' - 1"	60	0715-0815	600	74	+10	0	0 0	+ 5 0	8
				15	0730-0745	150	10	+10	0	0 0	+ 5 0	2
Gunston Rd & Meade Rd	West	Meade Rd (S)	11' - 7"	60	0700-0800	564	21	+10	12	3 0	+ 4 0	321
				15	0730-0745	141	7	+10	1	0.0	+ 5.0	155
Thoete Rd & 16th St	West	16th St	10' - 6"	60	0730-0830	486	37	+10	1	0 0	+ 5 0	51
				15	0745-0800	122	12	-10	1	0 0	+ 5 0	60
Humphreys Rd & Meade Rd	North	Humphreys Rd	12 - 0"	60	0700-0800	594	17	+10	NR	NR	+ 5 0	271
				15	0730-0745	149	7	+10	NR	NR	+ 5 0	88
Humphreys Rd & Meade Rd	East	Meade Rd (S)	10' - 0"	60	1630-1730	450	13	+10	337	T	+ 5 0	9
				15	1700-1715	113	2	+10	105	T	+ 5 0	4
Humphreys Rd & Meade Rd	South	Humphreys Rd	12' - 0"	60	1700-1800	594	20	+10	6	2 8	+ 4 0	NR
				15	1700-1715	149	5	+10	3	3.4	+ 3.0	NR
Abbot Rd & Willis Rd	East	Abbot Rd	10' - 4"	60	1630-1730	474	36	+10	NR	NR	+ 5 0	184
				15	1645-1700	119	15	+10	NR	NR	+ 5 0	58
Abbot Rd & Willis Rd	West	Abbot Rd	19' - 3"	60	0730-0830	1161	40	+10	33	10 5		NR
				15	0730-0745	290	6	+10	3	3 0	+ 4 0	NR
Abbot Rd & Willis Rd	South	Willis Rd (S)	10' - 3"	60	0700-0800	468	19	+10	165	T	+ 5 0	40
				15	0730-0745	117	4	+10	64	T	+ 5.0	6
Abbot Rd & Beauregard Rd	West	Abbot Rd	10' - 4"	60	0715-0815	474	45	+10	NR	NR	+ 5 0	37
				15	0730-0745	119	16	+10	NR	NR	+ 5.0	17
U S No 1 & Va No 235	North	Va No 235 (S)	20' - 5"	60	1600-1700	1245	10	+10	41	37 2	-10 0	68
				15	1615-1630	311	5	+10	16	45.7	-10.0	19
U S No 1 & Va No 618	East	U S No 1	19' - 0"	60	1630-1730	1134	56	+10	NR	NR	+ 5 0	132
				15	1645-1700	284	12	+10	NR	NR	+ 5 0	27
U S No 1 & Va No 618	West	U S No 1	20' - 0"	60	0700-0800	1215	54	+10	112	9 4	0 0	NR
				15	0730-0745	304	16	+10	40	8 4	+ 1 0	NR
U S No 1 & Va No 618	South	Va No 618	11' - 2"	60	0715-0815	534	14	+10	177	T	+ 5 0	47
				15	0745-0800	134	4	+10	58	T	+ 5.0	8
Humphreys Rd & U S No 1	West Thru	U S No 1	23' - 6"	60	1645-1745	1454	40	+10	NR	NR	+ 5 0	NR
				15	1700-1715	363	12	+10	NR	NR	+ 5 0	NR
Humphreys Rd & U S No 1	South Left	Humphreys Rd (S)	12' - 0" (8)	60	1630-1730	594	29	+10	NR	NR	+ 5 0	363
				15	1700-1715	148	7	+10	NR	NR	+ 5.0	122
U S No 1 & Va No 617	West	U S No 1	20' - 0"	60	1630-1730	1215	40	+10	163	24 4	- 7 0	NR
				15	1700-1715	304	4	+10	49	28 3	- 9 0	NR
U S No 1 & Va No 617	South	Va. No. 617 (S)	15' - 10"	60	0700-0800	834	13	+10	109	33 1	-10 0	221
				15	0730-0745	208	3	+10	57	41.0	-10.0	82
U S No 1 & Bache Rd	North	Bache Rd. (S)	10' - 0"	60	1615-1715	450		+10	98	42 6	-10 0	132
				15	1630-1645	113		+10	40	47.6	-10.0	44
Va No 617 & Va No 613	East	Va No 617	11' - 0"	60	0700-0800	522	21	+10	262	T	+ 5 0	175
				15	0730-0745	130	5	+10	93	T	+ 5 0	36
Va No 617 & Va No 613	South Right	Va. No. 613 (S)	10' - 6"	60	1645-1745	486	8	+10	248	83 0	-10 0	NR
				15	1645-1700	122	4	+10	91	88.0	-10.0	NR
Wills Rd & Va No 613	West	Wills Rd. (S)	11' - 6"	60	1630-1730	558	11	+10	84	26 3	- 8 0	235
				15	1645-1700	139	3	+10	23	22.1	- 6.0	81
Shirley Hwy & Va No 617	North	West Ramp (S)	13' - 9"	60	0700-0800	720	16	+10	422	T	+ 5 0	2
				15	0715-0730	180	4	+10	148	T	+ 5 0	0
Shirley Hwy & Va No 617	West	Va. No. 617	13' - 9"	60	1645-1745	720	11	+10	357	86 0	-10 0	NR
				15	1715-1730	180	3	+10	107	82.0	-10.0	NR
Humphreys Rd & U S No 1	West Rt to Gate	U S No 1	10' - 0" (9)	60	0700-0800	1000 (4)	13	0	(4)	(4)	(4)	(4)
				15	0730-0745	250 (4)	3	0	(4)	(4)	(4)	(4)

NP Not permitted
NR No road
S Stop sign
T T-Intersection
(1) Already converted to E P C in Col 23 shown for information only
(2) Maximum reduction for right turn is 10%
(3) Maximum reduction for left turn is 20%
Maximum combined reduction is 20%
E P C = equivalent passenger cars
(4) Added turning lane on separate signal 1000 vehicles/10 ft /hr (Ref p 91 para c, Highway Capacity Manual)

(5) Traffic backed up almost to Va No 235
(6) 3/4 hr 438/hr 1/4 hr 292 per 1/4 hr = 621
(7) Arbitrarily reduced because of gutter across right turn
(8) Only one lane of approach road is normally used because of (1) custom (2) lack of signing, (3) only one lane available on U S No 1 for eastbound traffic (other lane is used by traffic Ramp to South Gate)
(9) Width of approach governed by lane width of U S No 1
20' ramp width

ON-AVERAGE CONDITIONS					
Turn	No Parking or Bus Stop	Total	Adjusted Prac Capacity per Hr of Green	Go Time as Fraction of Period Minutes	Practical Capacity Under Local Conditions Pass cars/ Peak Pd
Adj (2) %	Adj %	Adj %			
		(9)-(12)-(15)-(16)	(7) x (17)		(18) x (19)
(15)	(16)	(17)	(18)	(19)	(20)
-12 3	+5	-3 8	623	23/60	239
- 7 0	+5	8 0	162	7/15	75
-20 0	+5	0 0	522	31/60	269
-20.0	+5	0.0	130	8/15	69
+ 6 4	+5	+17 0	723	46/60	554
+ 6 0	+5	+19 0	173	9/15	104
+ 5 0	+5	+25 0	743	30/60	372
+ 4 4	+5	+24 0	184	6/15	74
+10 0	+5	+15 0	959	15/60 Est	240
-20 0	+5	0 0	218	2/15	29
+10 0	+5	+30 0	549	27/60	247
+10 0	+5	+30.0	157	8/15	84
+10 0	+5	+30 0	772	48/60	618
+10 0	+5	+30 0	194	11/15	142
+ 5 0	+5	+25 0	698	42/60	489
+ 5 4	+5	+25 0	174	14/15	162
+10 0	+5	+26 0	816	58/60	788
+10 0	+5	+26 0	204	15/15	204
+10 0	+5	+15 0	1507	37/60	928
+10 0	+5	+15 0	389	5/15	129
+ 9 4	+5	+29 0	1671	42/60	1169
+10.0	+5	+30.0	421	8/15	224
(4)	(4)	0 0	1000	16/60	226
(4)	(4)	0 0	250	3/15	50
(4)	(4)	0 0	1000	42/60	700
(4)	(4)	0 0	250	8/15	133
+10 0	+5	+15 0	714	58/60	690
+10 0	+5	+15 0	336	13/15	291
-24 0	+5	- 5 0	540	54/60	486
-20.0	+5	0.0	142	12/15	114
- 6 0	+5	+12 0	706	58/60	682
- 6 0	+5	+13 0	179	14/15	167
+10 0	+5	+27 0	686	56/60	640
+10 0	+5	+28 0	173	11/15	127
-14 6	+5	- 5.0	564	57/60 Est	536
-14 0	+5	- 5.0	141	14/15 Est	132
-13.0	+5	+ 4.0	636	41/60	434
- 2 0	+5	+17 0	179	8/15	95
- 5 0	+5	+ 7.0	655	41/60	447
- 5 0	+5	+ 8 0	165	14/15	154
+ 6 0	-5	+11.0	579	44/60	425
+ 8 0	-5	+13 0	147	8/15	79
- 5 0	+5	0 0	522	31/60	270
+ 7.0	+5	+12.0	146	6/15	58
+ 6 0	+5	+21 0	670	39/70	373
+ 6 0	+5	+21 0	174	39/70	97
+ 4 0	+5	+18 0	693	39/70	386
+ 5 5	+5	+22 0	179	39/70	99
- 8 0	+5	+12 0	685	23/70	225
-10 0	+5	+10 0	168	23/70	55
-20 0	+5	- 4 0	553	23/70	181
-20.0	+5	- 4.0	138	23/70	45
+10 0	+5	+15 0	932	50/60	780
+10 0	-5	+15 0	232	12/15	189
+ 8 5	-5	+29 0	774	48/60	712
+ 9 0	+5	+29 0	194	10/15	129
-20 0	-5	- 1 0	558	30/60	279
-20.0	+5	0.0	141	5/15	47
- 6 0	+5	14 0	554	58/60 Est	535
-20 0	+5	0 0	122	15/15 Est.	122
-20 0	+5	0 0	594	57/60	564
-20 0	-5	0 0	149	14/15	139
+ 8 0	+5	+28 0	576	56/60	537
+ 6 0	+5	+26 0	142	13/15	123
+10 0	+5	+29 0	766	55/60	704
+10.0	+5	+28.0	191	10/15	128
-20 0	+5	0 0	474	54/60	427
-20 0	+5	0 0	119	14/15	111
+10 0	+5	+25 0	1451/1200 (7)	52/60	1039
+10 0	+5	+29 0	374/300	14/15	280
-10 0	+5	+10 0	515	39/60	335
- 1.0	+5	+21.0	142	9/15	85
+ 1 0	+5	+19 0	564	57/60 Est	536
0.0	+5	+20.0	143	14/15 Est.	133
+10 0	+5	+15 0	1431	7/60 Est	166
+10.0	+5	+15.0	358	2/15 Est.	48
+ 1 0	+5	+21 0	1372	40/60	915
+ 3 0	+5	+23 0	349	10/15	233
+10 0	+5	+25 0	1518	31/60	783
+10 0	+5	+26 0	383	7/15	179
-10 0	+5	+15 0	614	12/60	123
- 2.0	+5	+23.0	165	2/15	22
+10 0	+5	+30 0	1890	57/60	1796
+10 0	-5	+30 0	472	14/15	440
+10 0	-5	+30 0	772	38/60	489
+10.0	+5	+30 0	192	9/15	115
+10 0	-5	+18 0	1434	60/60	1434
+10 0	-5	+16 0	353	15/15	353
+10 0	0	+10.0	917	53/60	810
+10.0	0	+10.0	228	13/15	197
+10 0	+5	+15 0	518	50/60	431
+10.0	+5	+15.0	130	10/15	83
-20 0	-5	0 0	522	55/60 Est	478
-18 0	+5	+ 2 0	133	14/15 Est	124
+10 0	+5	+15 0	559	56/60	522
+10.0	+5	+15.0	140	14/15	131
+10 0	+5	+17 0	653	55/60 Est	598
+10.0	+5	+19.0	165	12/15 Est.	132
+10 0	+5	+30 0	936	57/60 Est	889
+10 0	+5	+30 0	234	13/15 Est	156
+10 0	+5	+15 0	828	60/60 Est	828
+10.0	+5.	+15.0	207	15/15 Est.	207
(4)	(4)	(4)	1000	60/60	1000
(4)	(4)	(4)	250	15/15	250

RESULTS

PEAK PERIOD ACTIVITY					
Length of Time	Maximum Desirable Flow	Actual Flow	Unused Capacity	Excess of Actual Flow over Desirable	
				Computed	Rounded
Minutes	Passenger Cars	E P C. (3)	%	%	%
(5)	(20)		(22)-(23) / (22)	(23)-(22) / (22)	
(21)	(22)	(23)	(24)	(25)	(26)
60	239	430		80	80
15	75	149		99	100
60	269	237	12		
15	69	82		19	20
60	554	749		35	30
15	104	243		134	130
60	372	634		70	70
15	74	229		209	210
60	240	233	3		
15	29	85		193	190
60	247	329		33	30
15	84	132		57	60
60	618	1125		82	80
15	142	336		137	140
60	489	987		102	100
15	162	346		114	110
60	788	956		21	20
15	204	286		40	40
60	928	196	79		
15	129	71	45		
60	1169	725	38		
15	224	268		20	20
60	226	172	35		
15	50	50		0	0
60	700	725		4	0
15	133	268		102	100 (5)
60	690	344	50		
15	291	291		0	0
60	486	218	55		
15	114	88	23		
60	682	391	43		
15	167	192		15	10
,0	640	269	58		
15	127	198		56	60
60	536	65	88		
15	132	22	83		
60	434	386	11		
15	95	181		91	90
60	447	292	35		
15	154	226		47	50
60	425	201	53		
15	79	102		29	30
60	270	154	43		
15	58	69		20	20
60	373	579		55	50
15	97	186		73	70
60	386	415		7	10
15	99	263		166	170
60	225	262		16	20
15	55	89		62	60
60	181	264		46	50
15	45	90		100	100
60	780	819		5	0
15	189	237		25	20
60	712	506	29		
15	129	158		22	20
60	279	362		30	30
15	47	169		260	260
60	535	321	39		
15	122	133		9	10
60	564	396	30		
15	139	118	15		
60	537	346	36		
15	123	109	11		
60	704	213	70		
15	128	86	33		
60	427	411	4		
15	111	131		18	20
60	1039	313	70		
15	280	97	65		
60	335	205	39		
15	85	70	18		
60	536	347	35		
15	133	167		26	30
60	166	110	31		
15	48	35	27		
60	915	1436		57	60
15	233	413		77	80
60	783	1189		52	50
15	179	474		165	160
60	123	224		82	80
15	22	66		200	200
60	1796	624	65		
15	440	246	44		
60	489	447	9		
15	115	146		27	30
60	1434	665	54		
15	353	173	51		
60	810	330	59		
15	197	139	29		
60	431	230	47		
15	83	84		1	0
60	478	437	9		
15	124	129		4	0
60	522	299	43		
15	131	103	31		
60	598	319	47		
15	132	104	21		
60	889	424	52		
15	156	148	5		
60	828	413	50		
15	207	131	37		
60	1000	424	58		
15	250	154	38		

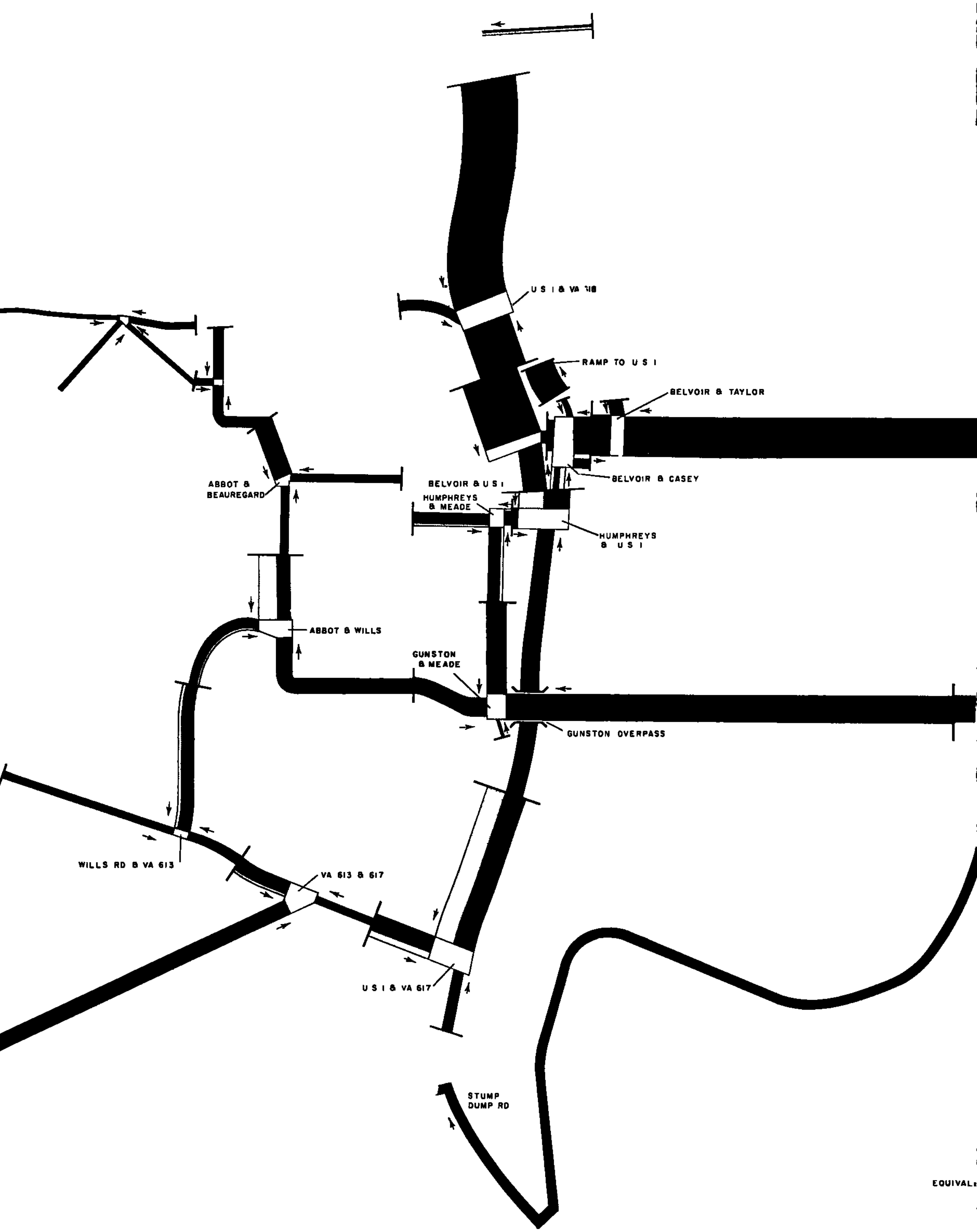
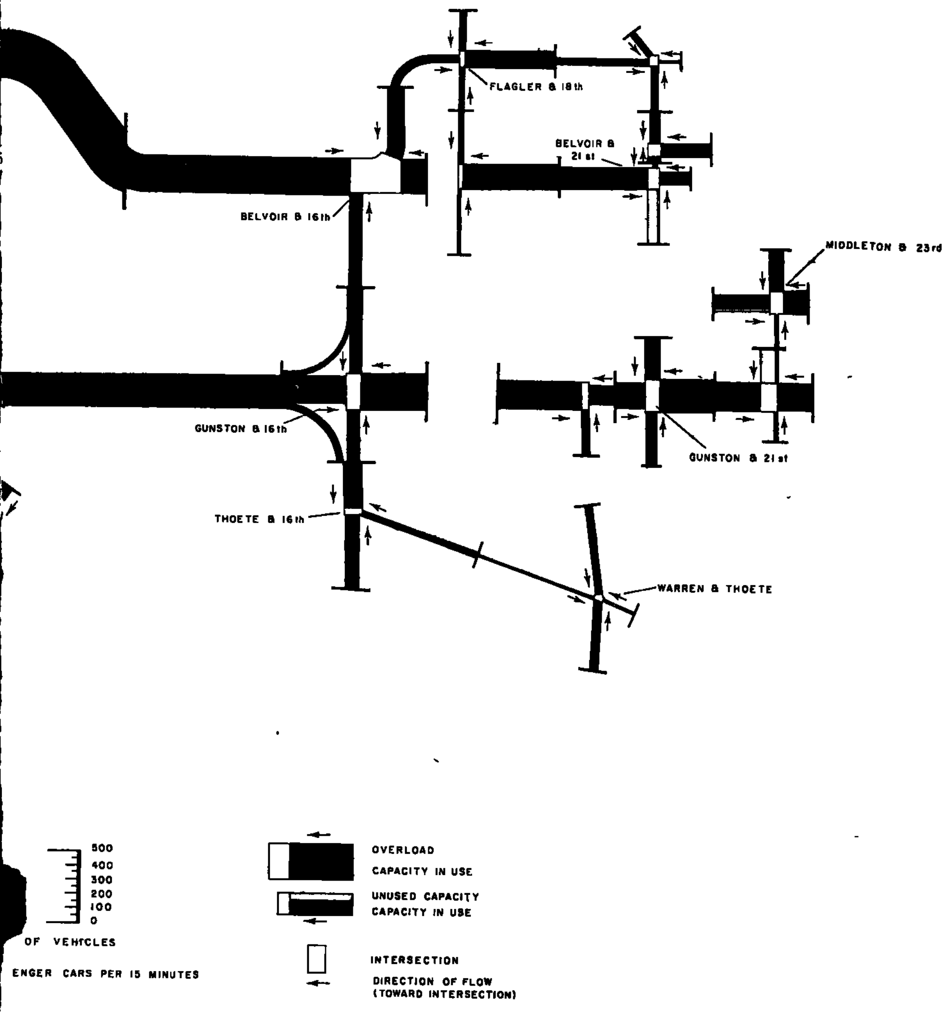


Figure 8. Peak-15-minute flows, capacities, overloads, and unused capacities on approach



anes of intersections, Fort Belvoir. Redrafted by Groll-Beach from original color plates.

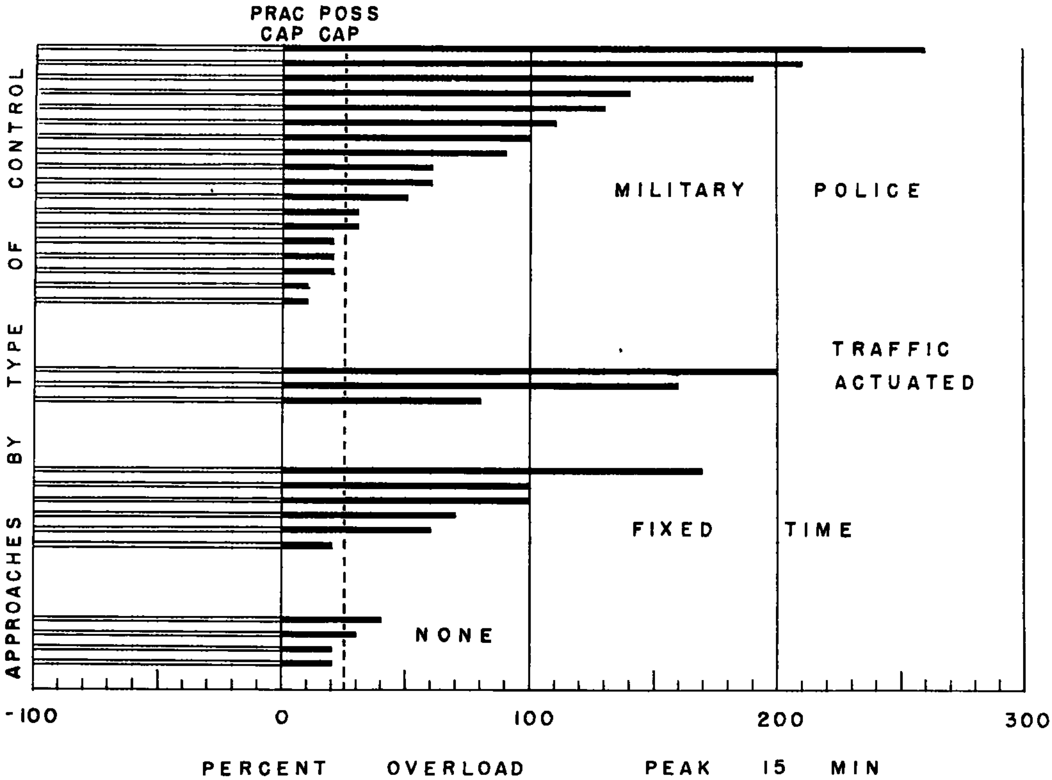


Figure 9. Peak 15 minute overloads on approaches to intersections, by type of control.

CONCLUSION

The principal conclusion which the author would draw from the Fort Belvoir Traffic Study lies in another field, however. While 31 intersection approaches were found to be overloaded in the peak 15 minute period, only two sections of roadway were found to be in urgent need of widening. Capacity restrictions in the post road net were due almost entirely to intersection deficiencies. Particularly on a military post, where all land is in single ownership and building setbacks are usually very generous, it is much simpler to increase road network capacity by enlarging intersections than by widening existing roads or building new roads. In general, it was surprising that budget and other authorities had allowed the traffic situation at Fort Belvoir to develop to the point it had reached, and that relatively simple remedial measures had not been taken.

But the lack of understanding of intersectional capacity limitations on military property is only a minor reflection of our more serious failure on a national scale to design and build the current additions to our urban road networks in such a way that intersection capacities will equal street capacities on primary and secondary uncontrolled access thoroughfares.

Do not the traffic engineers have a professional obligation to hammer away at state, county and city highway departments (who acquire land by purchase) and county and city planning commissions (who oversee the acquisition of land by dedication during the process of subdivision), insisting that right-of-way acquisition for intersections of major surface thoroughfares should be generous enough to provide additional lanes on the intersection approaches to replace the street capacity lost at the intersection through minutes of red, right and left turns, the slow starting of commercial vehicles, parking, and bus stops?

Discussion

HERBERT S. LEVINSON, Wilbur Smith and Associates, New Haven, Connecticut—Sutermeister has developed a most interesting analysis. His finding peak-hour volumes considerably in excess of published capacity values leads one to believe that the established capacity criteria cannot be universally or indiscriminately applied.

(1) Relating vehicular headways, effective lanes, and available green time, how would computed capacities compare with observed saturation loadings?

(2) If the roads at Ft. Belvoir were considered as "expressways," how would the saturation loadings compare with calculated capacity values using the Capacity Manual?

(3) Can it be inferred that, the capacity of any street - in vehicles per hour of green - lies somewhat between the expressway and typical street curves set forth in the Capacity Manual?

(4) Would the exact values to be used depend on the type and nature of marginal interferences resulting from abutting land use?

OSCAR SUTERMEISTER, Closure—In response to Mr. Levinson's questions:

(1) Headways were not timed in the field. Average headways could be computed from data presented on effective lanes, available green time, and actual flow. Capacity computed on the basis of such calculated average headway would of course equal observed saturation loading. If some normal or standard headway were used to compute capacity, the result would probably be somewhat lower than observed saturation loading.

(2) The suggested calculations and comparisons could be made from data presented, but have not been worked out by the author.

(3) In my opinion, no. Special conditions outlined in the paper differentiate Belvoir roads from usual city streets enough to preclude the inference that all streets have a capacity equal to or higher than that indicated in the Manual.

(4) Not necessarily, I feel. Departures from Manual capacity in the Belvoir case were not due to "type and nature of marginal interferences resulting from abutting land use." They were due to special local characteristics of traffic operations and of the traffic stream, as identified in the paper.

Capacities of One-Way and Two-Way Streets with Signals and with Stop Signs

ALEXANDER FRENCH, Highway Transport Research Engineer[1]
Bureau of Public Roads

● BOTH stop signs and signals are used extensively to control traffic at intersections. Generally, stop-sign control is used only on the less important of the two intersecting streets, but in some instances four-way stop-sign control is used. The relative merits of each of the various types of control devices has long been a topic of discussion and

TABLE 1

TRAFFIC VOLUMES AND DELAY PER VEHICLE WITH STOP-SIGN CONTROL ON ALL APPROACHES
While Volume on 13th Street was at a Peak

Location	1-Hour period						10-Minute period					
	For 13th Street		For cross streets		Total for intersection		For 13th Street		For cross streets		Total for intersection	
	Traffic volume	Avg. delay	Traffic volume	Avg delay	Traffic volume	Avg. delay	Traffic volume	Avg. delay	Traffic volume	Avg delay	Traffic volume	Avg. delay
	Vph	Min	Vph	Min.	Vph	Min	Vph	Min.	Vph	Min	Vph	Min.
13th Street two-way (parking one side)												
At Park Road	1,231	0 44	512	0 54	1,743	0.47	1,476	0.31	642	0 39	2,118	0 33
At Columbia Road	1,400	0 41	567	0.27	1,967	0 37	1,602	0 49	480	0.28	2,082	0 44
At Harvard Street	1,389	0.49	492	0 47	1,881	0 48	1,542	0.47	594	0 42	2,136	0.46
13th Street one way (parking prohibited)												
At Park Road	1,502	0 23	691	0 80	2,193	0.41	1,794	0 25	810	1 05	2,604	0 50
At Columbia Road	1,615	0.31	668	0 42	2,283	0 34	2,280	0.43	762	0.52	3,042	0 45
At Harvard Street	1,481	0 38	653	0 60	2,134	0 45	1,890	0.26	696	0 86	2,586	0 42
While Volumes on Two-way Cross Street were at a Peak												
13th Street two-way												
Park Road	1,147	0.35	565	0.54	1,712	0.41	1,476	0 31	642	0.39	2,118	0.33
13th Street one-way												
Park Road	1,502	0 23	691	0 80	2,193	0 41	1,794	0 25	810	1 05	2,604	0.50
While Volumes on One-way Cross Streets were at a Peak												
13th Street two-way												
Columbia Road Westbound	1,400	0 41	567	0.27	1,967	0.37	1,314	0 36	684	0.37	1,998	0 36
Harvard St Eastbound	1,387	0 53	495	0.49	1,882	0.52	1,530	0 41	600	0.36	2,130	0.40
13th Street one-way												
Columbia Road Westbound	1,615	0 31	668	0.42	2,283	0.34	1,968	0.45	822	0.70	2,790	0.52
Harvard St Eastbound	1,481	0.38	653	0.60	2,134	0.45	1,818	0.25	858	0.45	2,676	0.31

it is therefore desirable that factual information be obtained for use as a guide in determining the conditions under which traffic signals, two-way stop signs, and four-way stop signs provide the most efficient operation.

[1] The data were collected in the field by 19 Junior Highway Engineers. A preliminary report was prepared by four of these engineers, Robert D. Bee, Walter W. Bryant, Dwight A. Hodgens, Jr., and Joseph Rekas, as part of the Bureau of Public Roads Training Program. This report is based chiefly on further analysis of the data. The District of Columbia Department of Vehicles and Traffic supplied the signs and adjusted signal timing as directed by John H. Mitton, Assistant Director and Traffic Engineer. The Metropolitan Police provided officers for emergency control as directed by Deputy Chief of Police John J. Agnew.

This report covers a study of traffic at four intersections while the traffic signals were in normal operation and also studies at the same locations while two-way stop signs and four-way stop signs were used in place of the traffic signals.

All four intersections are on 13th Street, N.W., in Washington, D.C., a four-lane street which, during the peak period carries more than 3,200 vehicles per hour. It is operated one-way inbound during the morning peak period and one-way outbound during the evening peak period. During off-peak periods, it operates as a two-way street with parking on one side. The 1.8-mile section of 13th Street between Logan Circle and Spring Road with a peak-hour overall speed of about 18 mph is an unusually efficient arterial street, carrying far heavier traffic than most streets of this width are capable of accommodating under usual conditions. The three types of intersection control were observed under heavy traffic loads at intersections with both one-way and two-way streets.

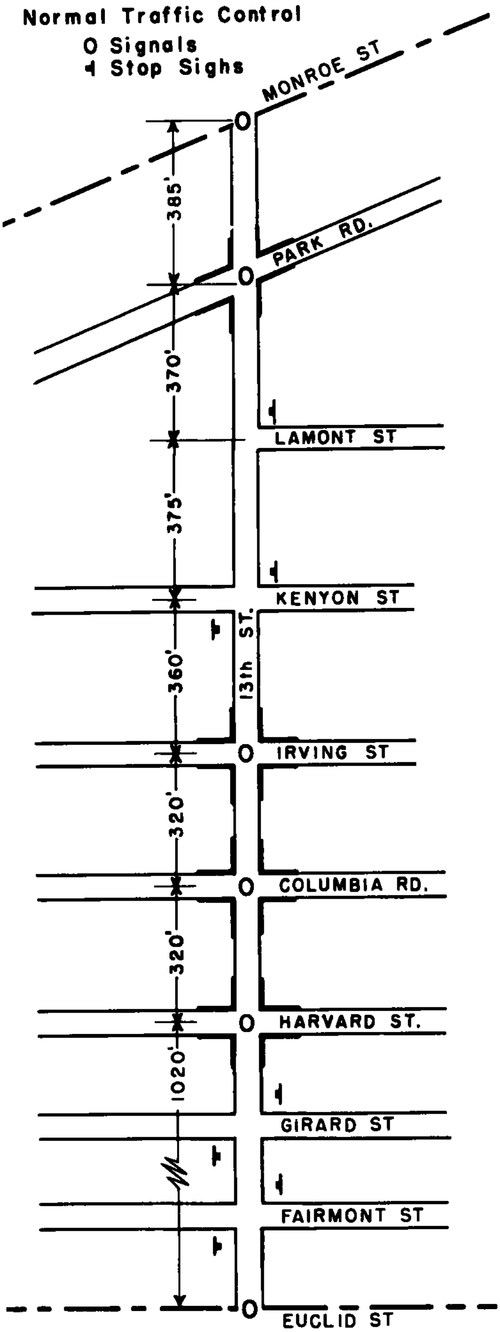


Figure 1. Four intersections studied on Thirteenth Street.

Thirteenth Street is a through north-south street carrying much heavier traffic volumes than the cross streets. The cross streets at which studies were made are: Irving Street and Park Road, which carry two-way traffic; Harvard Street, which is one-way eastbound; and Columbia Road, which is one-way westbound. The general layout is shown in Figure 1. Details of each intersection are shown in Figure 2, while Figures 3 and 4 are photographs of 13th Street when operating two-way and one-way respectively.

All four intersections are normally controlled by an interconnected system of coordinated fixed-time traffic signals. There is a single dial in the time controller at each intersection. Consequently, the 80-second signal cycle and the stop and go intervals for 13th Street and the cross streets remain constant throughout the day. The signals are interconnected with a master controller, however, so that the time offsets between successive signals on 13th Street can be changed. This provides for progressive traffic movement favoring the desired direction of travel at different times of the day.

Three different signal progressions are used. The first, which is used during the morning rush period, provides progressive movement for 13th Street traffic while it is one-way inbound. The second, which is used during nonrush periods, provides reasonably good progression in both directions while 13th Street carries two-way traffic. The third, which is used during the afternoon rush, provides excellent progression while 13th Street is operating one-way outbound.

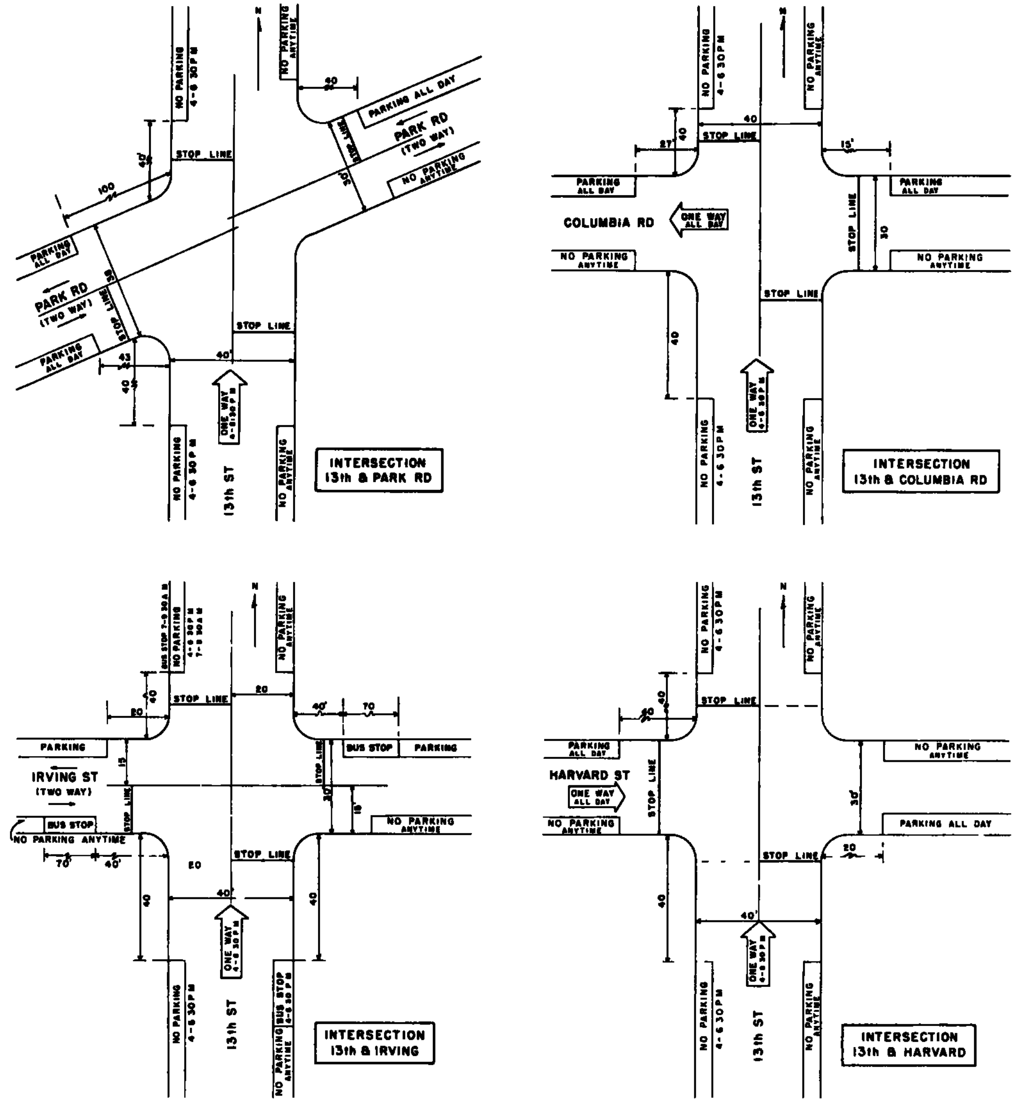


Figure 2. Details of intersections studied.

Accident data for 13th Street are summarized and compared with those for other arterial streets and a nearby freeway in Table 2. The accident rate is 13 percent higher than the average based on a nationwide study of representative arterial city streets. This higher accident rate may be due in part to the very complete reporting of accident data in the District of Columbia. It is not believed to indicate a significantly higher accident potential. It is much higher, however, than the accident rate for the Shirley Freeway, a facility with full control of access which carries comparable traffic volumes.

The principal field studies were conducted on three weekday afternoons in March 1954. Operation with signal control was observed the first day. The green time on the cross streets was reduced in an attempt to provide a continual backlog of vehicles on cross streets so that their possible capacities could be determined. On another day, stop signs were used to control traffic on the cross streets. On a third day, stop-sign control was used on all approaches to the intersections. Additional field studies were conducted in November to measure the delay to traffic with the normal signal timing and to determine the capacity of 13th Street with two-way traffic controlled by signals.

TABLE 2

ACCIDENTS PER HUNDRED MILLION VEHICLE-MILES ON THIRTEENTH STREET COMPARED WITH OTHER FACILITIES

	13th Street[a]	Arterial streets[b]	Shirley Freeway[c]
All reported accidents	1,091	966	192
Fatalities	2	3	3

[a]Logan Circle to Spring Road 1.8 miles, January 1, 1952, through June 30, 1954.
[b]Preliminary figure for urban streets with no access control based on cooperative study by the Bureau of Public Roads and State Highway Departments.
[c]Ridge Road to Shirlington Circle 1.6 miles, Arlington County, Virginia, January 1, 1950, through June 30, 1954.

Manual counts of all through and turning movements, classified by type of vehicle, were made at all entrances to the intersections. These were recorded for every cycle when signals were in operation, and every 2 minutes when signals were off and traffic was being controlled by stop signs. Fully utilized "go" periods (loaded cycles) were noted while observing traffic controlled by signals. To provide data from which vehicular delay might be computed, counts were made of standing vehicles on each intersection approach at regularly spaced intervals. Thirty-second intervals were used when signals were in operation and two-minute intervals were used when stop-sign control was employed.

Field data have been summarized by 10-minute periods. Rates per hour of green signal time have been calculated so that the rates are comparable regardless of signal timing. The rate per hour of green is calculated by dividing the hourly volume as normally determined by the percentage that the green period, not including any amber time, is of the total cycle.

To calculate total delay, the number of stopped vehicles as determined by the periodic count was multiplied by the time interval between counts. This value was then divided by the number of vehicles entering the intersection to determine the delay per vehicle.

SIGNAL CONTROL

Despite the high traffic volumes observed at the four intersections while they were controlled by signals, the traffic demand on some of the approaches was insufficient to utilize fully the green interval during any of the signal cycles. On several of the approaches the green interval was fully utilized during a few cycles only. The green or "go" interval was considered fully utilized when the traffic demand at the observed approach was equal to the possible capacity of that intersection approach with vehicles continuously entering the intersection throughout the green interval. Cycles during which these conditions occurred are called "loaded cycles." A loaded cycle for an intersection approach is independent of the traffic on the other approaches which may or may not be loaded during the same period.

In Table 3 the first four columns show the traffic on 13th Street, the percentage of dual-tired vehicles, and the percentage of right and left turns during the 1-hour period of maximum traffic. The next column shows the number of seconds of green signal time per cycle. The traffic volume per hour of green signal time follows. The highest hourly rate observed during 10 consecutive minutes is shown next, followed by the rate per hour of green time for loaded cycles. All these data were compared for similar intersection approaches, and the possible capacity was determined for each approach. This is tabulated in the last column of the table. Table 4 is similar to Table 3 and lists the data for the cross streets.

The estimated possible capacities shown in the last columns of Tables 3 and 4 were determined on the basis of the 1-hour, 10-minute, and loaded-cycle volumes. Data for similar intersection approaches were considered collectively in arriving at these capacities. The values represent the possible capacities over a 1-hour period even though volumes for shorter time periods were used in their determination. A detailed description of the method of determining these values is necessary for an appreciation of their reliability.

Figure 3. Looking south on Thirteenth Street during off-peak period - near Irving Street with two-way traffic on Thirteenth Street.

TABLE 3

OBSERVED TRAFFIC AND POSSIBLE CAPACITY OF 13th STREET[a] AT FOUR SIGNAL-CONTROLLED INTERSECTIONS

13th Street Carrying Two-way Traffic

Intersection entrance	Conditions during continuous 1-hour period of maximum traffic: Volume	Dual tired	Left turns	Right turns	Green time for 80-sec. cycle	Maximum observed traffic vols. per hour of green or "Go" time: During a 1-hour period	During a 10-min. period	During loaded cycles[b]	Estimated possible capacity per hour of green
	Vph	Percent	Percent	Percent	Seconds	Vehicles	Vehicles	Vehicles	Vehicles
Southbound (parking permitted)									
At Park Rd. (two-way)	482	3.5	0.4	7.0	38	1,015	2,080	1,530(1-1)	1,600
At Irving St. (two-way)	524	2.9	1.9	1.3	-[c]	1,435	1,720	1,605(25-39)	1,600
At Columbia Rd. (one-way WB)	557	3.4	-	6.8	45	990	1,181	None	1,600
At Harvard St. (one-way EB)	555	4.3	5.4	-	29	1,531	1,707	1,610(55-78)	1,600
Northbound (parking prohibited)									
At Park Rd. (two-way)	836	2.7	4.1	6.6	40	1,672	2,124	2,460(3-3)	2,600
At Irving St. (two-way)	748	2.9	4.3	1.3	-[c]	2,100	2,916	2,635(10-25)	3,000
At Columbia Rd. (one-way WB)	671	3.3	6.8	-	45	1,193	1,611	None	3,000
At Harvard St. (one-way EB)	849	2.8	-	4.1	29	2,342	3,321	3,576(5-20)	3,300
13th Street Carrying One-way Traffic									
Northbound (parking prohibited)									
At Park Rd. (two-way)	2,747	0.9	6.4	10.3	40	5,494	7,140	6,857(8-16)	7,000
At Irving St. (two-way)	3,128	1.0	1.8	0.5	52	4,811	5,623	None	7,000
At Columbia Rd. (one-way WB)	3,168	1.0	3.6	-	45	5,633	6,614	None	7,000
At Harvard St. (one-way EB)	2,987	0.8	-	4.7	50	4,779	5,309	5,637(4-16)	7,000

[a]13th Street is 40 ft. wide. Parking is permitted only on the west side used by southbound traffic while 13th Street is operating two-way. All parking is prohibited on 13th Street while one-way.

[b]The number of loaded cycles observed is shown by the first of the two figures in parentheses while the second figure is the total number of consecutive cycles in the period during which the loaded cycles were observed.

[c]24 32-second green periods, 11 28-second green periods, and 10 24-second green periods.

Figure 4. Looking south on Thirteenth Street during evening peak period - near Park Road with one-way traffic on Thirteenth Street.

TABLE 4

OBSERVED TRAFFIC AND POSSIBLE CAPACITY OF FOUR SIGNAL-CONTROLLED STREETS CROSSING 13th STREET[a]

Cross Streets Carrying Two-way Traffic

Intersection entrance	Conditions during continuous 1-hour period of maximum traffic					Maximum observed traffic vols. per hour of green or "Go" time			Estimated possible capacity per hour of green
	Volume	Dual tired	Left turns	Right turns	Green time for 80-sec. cycle	During a 1-hour period	During a 10-min. period	During loaded cycles[b]	
	Vph	Per-cent	Per-cent	Per-cent	Seconds	Vehicles	Vehicles	Vehicles	Vehicles
13th Street two-way									
Park Rd. EB	273	8.0	2.9	9.9	18	1,213	1,502	1,333(24-40)	1,400
Park Rd. WB	294	3.4	25.8	1.7	18	1,307	1,551	1,400(30-45)	1,400
Irving St. EB	203	13.8	13.8	18.7	18	902	1,200	1,120(10-44)	1,200
Irving St. WB	115	15.6	6.1	7.8	18	511	649	800(1-1)	1,200
13th Street one-way									
Park Rd. EB	366	2.7	13.1	-	18	1,627	1,849	1,542(43-52)	1,600
Park Rd. WB	304	3.0	-	4.9	24	1,013	1,249	1,433(6-52)	1,400
Irving St. EB	245	9.8	19.6	-	18	1,089	1,450	1,055(40-92)	1,400
Irving St. WB	179	7.3	-	16.8	18	795	1,000	850(8-39)	1,400
Cross Streets Carrying One-way Traffic									
13th Street two-way									
Columbia Rd. WB	465	3.9	7.3	9.0	15	2,480	2,917	2,824(13-28)	3,000
Harvard St. EB	447	5.6	20.4	5.1	15	2,384	2,821	3,120(5-7)	3,000
13th Street one-way									
Columbia Rd. WB	668	1.0	-	12.6	-[c]	3,185	3,960	3,553(41-95)	3,600
Harvard St. EB	708	1.8	33.5	-	17	3,332	3,936	3,600(60-89)	3,600

[a]All cross streets are 30 feet wide with parking permitted on one side except that the west leg at Park Road is 38 feet wide with parking permitted on both sides.

[b]The number of loaded cycles observed is shown by the first of the two figures in parentheses while the second figure is the total number of consecutive cycles in the period during which the loaded cycles were observed.

[c]37 15-second green periods and 8 25-second green periods.

First, the 1-hour volume was compared with the maximum 10-minute volume and the volume during loaded cycles at the particular approach. The relation between these values, as well as the frequency of loaded cycles, formed a basis for tentatively estimating possible capacity. For instance, if the 1-hour volume and loaded-cycle volume were well below the maximum 10-minute volume, it was evident that the first two were well below capacity. The presence of only a few loaded cycles during a large number of consecutive cycles would confirm this. In such a case it is possible that even the maximum 10-minute volume was below possible capacity. On the other hand, if the three volumes were about equal and a large and fairly concentrated number of loaded cycles were observed, this was an indication that possible capacity was reached. In such instances a volume somewhat below the maximum 10-minute volume, but not less than the loaded-cycle volume, might have been tentatively selected as the possible capacity.

After the tentative values for all similar intersection approaches were selected by this process, they were compared with each other for consistency and reasonableness. In this comparison differences in turning movements, frequency of commercial vehicles, bus stops, and other factors known to affect traffic operation were considered. If the tentative possible-capacity estimate for any approach appeared inconsistent, a reappraisal was made. In the case of similar approaches where the tentative estimates were close, a single value was selected for the possible capacity of all. If only one of several otherwise similar approaches was observed at or near possible capacity conditions, the value determined for this approach was assigned to the others.

Unusually high values were determined for the possible capacity of 13th Street when it was operating one-way northbound. The highest volumes per hour of green were observed at Park Road where the green time was least. Even here, with only one-half the total cycle green to 13th Street, only eight loaded cycles occurred. With a volume during these cycles of 6,857 vehicles per hour of green and a maximum 10-minute volume of 7,140 vehicles per hour of green, the indicated 7,000 vehicles per hour of green is evidently the approximate possible capacity. Since the turning movements were less at the other approaches, it must be inferred that possible capacities at these approaches are at least equal to the 7,000 vehicles per hour of green determined at Park Road. In the absence of factual data indicating higher capacity, this value was accepted for all four locations, rather than some higher value.

The results of an analysis of the data in Tables 3 and 4 indicate that within the range of values observed, the percentage of commercial vehicles and the percentage of left and right turning traffic at the intersections had little apparent effect on capacities of the approaches. An exception is the case of Irving Street which carried about twice as high a percentage of dual-tired vehicles as the other cross streets. It had a capacity somewhat lower than the other cross streets.

The rates of traffic flow based on the 10-minute periods of maximum volume were frequently greater than the rates based on the loaded cycles. These 10-minute periods included many cycles that were not fully loaded. It is evident, therefore, that greater volumes are sometimes carried during unloaded cycles than during loaded cycles. A cycle was considered to be loaded at an intersection approach if vehicles were traveling through the intersection during the entire green signal period and the stream was interrupted by the red signal. Frequently, when all drivers were alert, accelerated quickly, and allowed a minimum headway, all vehicles waiting at the intersection and those arriving during the green interval were able to clear the intersection before the green time expired. A cycle of this type would not be classed as a loaded cycle. At other times one or two slow drivers or some other impediment tended to reduce the movement of vehicles through the intersection and, as a result, the cycle was classified as being fully loaded despite the fact that the number of vehicles counted was less than during other cycles which were not so classified.

The possible capacity of 13th Street operating one-way northbound is 7,000 vehicles per hour of green. This is equivalent to 1,750 vehicles per hour of green per 10 feet of width or per lane. This extremely high volume is only 2 percent below the maximum 10-minute volume observed on 13th Street at Park Road. At the time there was little delay to traffic, although some vehicles required more than one signal cycle to clear the intersection. Traffic counts obtained by automatic recorders indicate that traffic volumes of 3,200 vehicles during 1 hour occur frequently. This is equivalent to 6,400 vehicles per hour of green at Park Road. Thus 13th Street often carries a volume for a 1-hour period which is in excess of the peak volume recorded during this study.

The possible capacity of 3,600 vehicles per hour of green for each of the one-way streets crossing 13th is also unusually high. This is equivalent to 1,200 vehicles per hour of green per 10 feet of surface width, curb to curb, with parking on one side. The capacity of each of the two-way streets crossing 13th Street is also unusually high.

These very high capacities certainly exceed those attained on most city streets. At these high capacities the flow of traffic through an intersection is likely to be greatly affected by a slight change in signal timing, especially in the time offset between succeeding signals which is very critical in a progressive system. Minor accidents, stalled vehicles, and severe weather conditions also result in sharp reductions in capacity. Capacity values considerably lower than those found on 13th Street must therefore be used for design when planning one-way street systems and other improvements to city traffic facilities, or it is likely that adequate capacity will not be provided.

By comparing the possible capacities of the approaches, the effect that two-way and one-way operation have on capacity has been determined for both 13th Street and the cross streets. The effect of parking on the capacity of 13th Street while operating as a two-way street was also determined by comparing the capacity in the northbound direction with the capacity in the southbound direction. Southbound vehicles could park on the west side of the street while parking was prohibited on the east side. The effects of these conditions are shown graphically in Figure 5.

Figure 5 shows that under one set of conditions the possible capacity of 13th Street is only 3,200 vehicles per hour of green signal time, whereas, under another set of conditions, the possible capacity is 7,000 vehicles per hour of green or 2.19 times as high as for the first. The improvement in capacity was realized by eliminating parking and changing from two-way to one-way operation on 13th Street. Likewise, the capacity of a cross street in one case is 2,600 vehicles per hour of green time and in another case 3,600 vehicles per hour of green. The difference of 1,000 vehicles per hour or 38 percent is a direct result of changing from two-way to one-way operation on both of the intersecting streets and does not involve a change in the parking condition on the cross street.

One of the most important results of the study, which is illustrated by Figure 5, is that the operating conditions on one street affect not only the capacity of that street but also the capacity of the intersecting street. Under certain conditions, for example, the capacity of 13th Street is higher when the cross street carries one-way traffic than when it carries two-way traffic. The same is true for the capacity of the cross streets in relation to the directional flow on 13th Street.

Table 5 shows the effect on intersection capacity of each change that was made on 13th Street and the cross streets with respect to directional operation and parking conditions. In this table 13th Street is referred to as the major street and the cross streets are referred to as minor streets. By using this terminology the results can be more directly compared with the results of studies at other locations and more readily applied to similar situations elsewhere.

TABLE 5

INCREASE IN CAPACITIES OF STREETS AT SIGNALIZED INTERSECTIONS BY CHANGING FROM TWO-WAY TO ONE-WAY OPERATION AND BY THE ELIMINATION OF PARKING ON MAJOR STREET[a]

Change in conditions	Increase in possible capacity: On major street (Percent)	Increase in possible capacity: On minor street (Percent)
Changing from two-way to one-way operation		
On major street with no parking		
With two-way traffic on minor street	25	12
With one-way traffic on minor street	11	20
On minor street		
With two-way traffic on major street		
Parking both sides of major street	0	15
Parking one side of major street	8	15
No parking on major street	12	15
With one-way traffic on major street		
No parking on major street	0	24
Elimination of parking from major street[b]		
From one side		
At one-way minor streets	48	-
At two-way minor streets	37	-
From both sides		
At one-way minor streets	97	-
At two-way minor streets	75	-

[a]Parking on one side of minor street in all cases
[b]Two-way traffic on major street in all cases

It is of particular interest that a change from two-way to one-way operation of traffic on the major street increased the capacity of the minor street as well as the capacity of the major street. The increase for the major street was 25 percent at locations where the minor street was two way and only 11 percent where the minor street was one-way. This same change also increased the capacity of the one-way minor streets 20 percent and the capacity of the two-way minor streets 12 percent. Thus, the minor streets that were benefited most were those at locations where the major street benefited least by a change in its directional operation.

The cross streets with one-way traffic had a greater capacity than those with two-way traffic. The difference was greater (24 percent compared to 15 percent) when the major street was also one-way rather than two-way. This difference in operation did not, however, benefit the capacity of the major street when it carried two-way traffic with parking on both sides or one-way traffic with no parking. One-way operation on the cross streets was of some benefit to the major street when it carried two-way traffic and parking was eliminated from one or both sides.

Eliminating parking on the major street had a far greater effect on its capacity than changing from two-way to one-way operation. Eliminating parking from both sides of the major street nearly doubled its capacity (an increase of 97 percent) at the intersections with one-way minor streets. At intersections with two-way minor streets, the increase was 75 percent. Eliminating parking on one side of the major street had about half the effect of eliminating parking on both sides.

It should also be pointed out that progressive movement, which reduces travel time, is usually more easily attained on a one-way than on a two-way street. This is an additional and very important advantage of one-way operation.

STOP-SIGN CONTROL ON CROSS STREETS

All cross streets were operating at their possible capacities during the rush period on the day that stop signs were used only on the cross streets. Long queues of waiting vehicles developed, and occasionally it was necessary for a police officer to clear the backup on the cross streets. Vehicles on the cross streets were aided somewhat by the signals at intersections on the through street north and south of the study area, which caused gaps

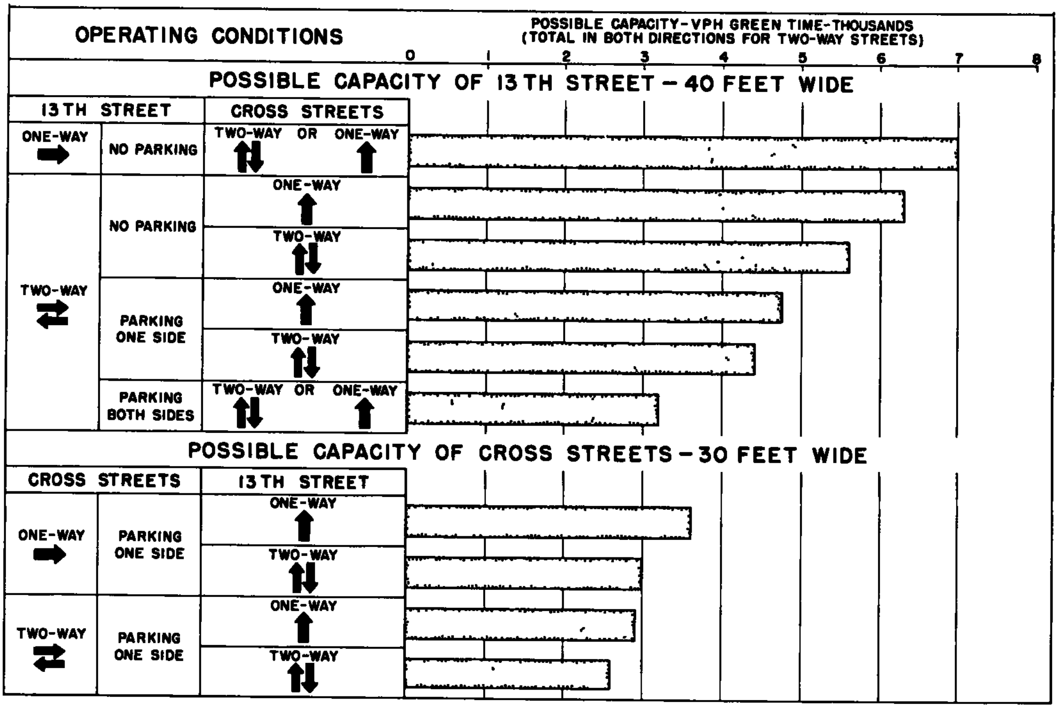


Figure 5. Street capacity with signal control related to directional operation and parking conditions.

in traffic on 13th Street.

Table 6 shows the observed traffic volumes and the delays to traffic on the cross streets at the intersections studied. Volumes and delays are listed for the 1-hour and for the 10-minute periods of heaviest cross-street traffic while 13th Street was one-way and also while 13th Street was two-way.

The same type of information was obtained for all 10-minute periods during the studies, and the average delays to cross-street traffic were compared for various traffic volumes on the cross streets and on 13th Street. As was expected, the delay to cross-street traffic was found to increase with an increase in the total traffic volume on 13th Street as well as with an increase in the total cross-street traffic. A less expected finding was that the delay to cross-street traffic was independent of the directional usage of either 13th Street or the cross street. Consequently, the delay to cross-street traffic when controlled by stop signs can be shown on one graph representing both two-way and one-way operation on both of the intersecting streets. This has been done in Figure 6.

The two lower curves of Figure 6 show the combinations of sustained through-street and stop-street traffic volumes which produced average delays to cross-street traffic of 30 seconds and of 1 minute per vehicle. The total traffic volume on 13th Street is shown on the horizontal axis, and the total traffic volume on the cross street is shown on the vertical axis. For example, a cross flow of about 600 vehicles per hour can be accommodated with an average delay of 1 minute when the total traffic volume on 13th Street is 1,600 vehicles per hour. Combinations of volumes which lasted for only short periods sometimes caused delays much longer or shorter than those indicated by the curves. If the volumes were sustained for 20 or 30 minutes, however, the delays were as indicated by the curves.

The third curve in Figure 6 shows the maximum volumes that the cross streets can accommodate with various volumes on 13th Street. The delay accompanying these volumes was at least 2 minutes and for any given combination of volumes might have been several minutes. Once the traffic volumes on the cross street and on 13th became sufficiently great to cause a delay of 2 minutes, the delay could increase to several minutes within a short period of time with no change in the traffic volumes. This curve therefore also represents the traffic volumes for all delays above 2 minutes. It thus represents the possible capacities of the cross streets when controlled by stop signs and with no control on 13th Street.

Volumes which cause a 30-second delay can be exceeded by as much as 100 percent without increasing the delay to more than 1 minute. Volumes which result in a 1-minute average delay, however, can only be exceeded by about 10 percent without increasing the delay to more than 2 minutes.

The small difference between the volumes shown for the curve representing a 1-minute delay and the curve representing a delay in excess of 2 minutes indicates that at these intersections the critical volumes are those producing a delay of between 1 and 2 minutes per vehicle to cross-street traffic.

The volume combinations indicated by the 30-second delay curve fit the usual definition of practical capacity for the prevailing roadway and traffic conditions since greater delay and restriction to movement would appear unreasonable to most drivers. With the normal setting of the signals, average delays of 30 seconds or more to minor-street traffic were infrequent even at the highest volumes observed. In another study it was found that drivers are unwilling to accept longer delays at stop signs than at signals.[2] It is concluded that the practical capacity of the cross street when controlled by stop signs with no control on the through street is represented by the curve for an average delay of 30 seconds.

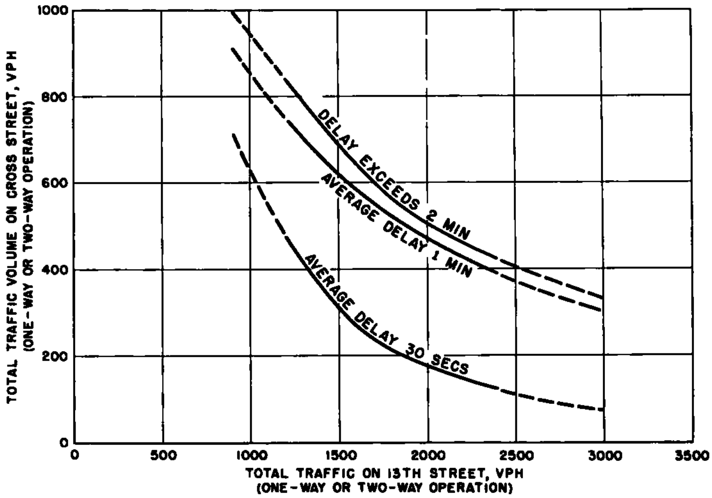


FIGURE 6 - DELAY TO TRAFFIC ON CROSS STREETS WHEN CONTROLLED BY STOP SIGNS (NO CONTROL ON 13TH ST)

Figure 6. Delay to traffic on cross streets when controlled by stop signs (no control on Thirteenth Street).

Figure 7 shows the possible and practical capacities of the cross streets with stop-sign control on these streets and the possible capacities with signal control. The curves for stop-sign control represent both one-way and two-way operation and are based on the delay to cross-street traffic. With stop signs on the cross streets, 13th Street could no doubt carry as much traffic as with signals but not without an unreasonable delay to cross-street traffic.

For the signals, separate curves are shown for one-way and two-way operation, but separate curves are not shown for possible and practical capacities. With the progressive signal control there was no appreciable increase in the delay to traffic as the traffic increased from comparatively low volumes to those at possible capacity. Thus it was not feasible to establish a value for practical capacity with progressive signal control on the basis of delay to traffic. For the purpose of comparing the three different types

[2] "Effects of Reversible Lane Movement, Signalization of Three-Lane Highways," by M. Mansfield Todd. Proceedings HRB 1950.

of control on the basis of a tolerable delay to traffic, the curves for practical capacity with stop-sign control should be compared directly with the possible capacity curves for signal control.

It may be noted that the curves for signal control intersect the x axis and the y axis at values below those given in the last columns of Tables 3 and 4. This is because the amber time in each cycle was not included in the green time when calculating the traffic volumes for the signal-control curves.

From Figure 7 it may be seen that both the possible and the practical capacities of the cross streets are much lower with stop-sign control than with signal control. This is especially true with one-way operation on both streets. Even with volumes as low as 1,000 vehicles per hour on 13th Street and with two-way traffic on both streets, the possible capacity of the cross streets with stop signs is about one-half the capacity with signal control. With one-way traffic and at other volumes on 13th Street, the difference is even greater.

At all traffic volumes there was practically no delay to traffic on the through street when stop signs were used to control traffic on the cross streets. Cross-street traffic, however, experienced some delay even at the lower volumes, and this delay increased rapidly as the volume increased. With signal control both 13th Street traffic and cross-street traffic experienced some delay. This delay at all observed volumes, however, was never greater for the cross streets than the delay with stop signs. The delay with signals was small as a result of the well-coordinated progressive system operating on the cross streets as well as on 13th Street.

While the difference in delay with stop signs and with signals was small at low volumes, it became very large at the higher volumes. For example, when the traffic volume on the through street approached 3,000 vehicles per hour, the delay to traffic on the cross street varied from 1½ to 6 minutes per vehicle with stop signs on the cross street, compared to 20 to 40 seconds per vehicle with signals. At a volume of 1,800 vehicles per hour on the through street and a cross-street volume of about 250 vehicles per hour, the average delay to all traffic on both streets was 10 seconds per vehicle with cross-street traffic controlled by stop signs and also when traffic was controlled by signals. At lower volumes the delays were only slightly less and were about equal

TABLE 6

OBSERVED VOLUMES AND DELAYS FOR TRAFFIC CONTROLLED BY STOP SIGNS AT APPROACHES TO 13th STREET (NO CONTROL ON 13th STREET)

Cross Streets Carrying Two-way Traffic

Cross street	Study time[a]	Average for study			Average during peak 10 minutes		
		Traffic volume			Traffic volume		
		Cross street	13th street	Delay per vehicle on cross street	Cross street	13th street	Delay per vehicle on cross street
Name	Minutes	Vph	Vph	Minutes	Vph	Vph	Minutes
13th Street two-way							
Park Rd. EB	32	272	1,140	0 34	336	1,128	0.43
Park Rd WB	32	248	1,140	0.48	276	1,140	0.61
Irving St. EB	38	207	1,265	0 41	234	1,236	0 15
Irving St WB	38	111	1,265	0 46	114	1,236	0.74
13th Street one-way							
Park Rd EB	60	322	1,658	2 89	366	1,950	3 21
Park Rd. WB	60	228	1,658	0 51	276	1,746	0 74
Irving St EB	60	245	2,172	1 67	318	3,126	1.25
Irving St. WB	60	143	1,927	0.80	168	2,490	0 71
Cross Streets Carrying One-way Traffic							
13th Street two-way							
Columbia Rd. WB	40	491	1,270	0 59	570	1,260	0 63
Harvard St. EB	34	434	1,200	0.63	462	1,152	0.70
13th Street one-way							
Columbia Rd. WB	60	572	1,915	3 81	738	1,866	4 78
Harvard St EB	60	744	1,785	0 92	882	2,070	0 64

[a]Where study was conducted for more than 1 hour, the data for the 1-hour period of maximum flow have been used. The studies of less than a full hour are for the periods immediately ahead of the time when 13th Street was changed to one-way operation.

for both types of control. At volumes above 1,800 vehicles per hour on the through street, the average delay to all traffic was greater with stop-sign control on the cross streets than with signal control.

Traffic on the through street was protected by stop signs on the cross streets, and had it not been impeded at other intersections along the route, the volume of traffic could theoretically have increased to 7,000 vehicles per hour. This type of control is not feasible, however, for such heavy volumes of traffic on the through street because cross traffic is almost completely blocked.

When an intersection at which traffic on one of the streets is controlled by stop signs is adjacent to an intersection at which traffic on the through street is controlled by signals, the capacity of the cross street is somewhat greater than it would be if there were no traffic signals in the immediate vicinity. The signal at the neighboring intersection creates gaps in the stream of traffic on the through street into which vehicles waiting at the stop sign may enter. Traffic volumes greater than those indicated by the curves in Figure 7 could probably be discharged from cross streets controlled by stop signs at such locations.

The traffic volumes observed in this study are evidently too great to be handled satisfactorily at intersections controlled by stop signs on the cross street. Even with moderately heavy volumes on the cross streets, this traffic was delayed as much or more by stop signs than by signals. At the higher volumes the delay with stop signs definitely became intolerable to cross-street traffic, although through traffic was completely unrestricted.

STOP-SIGN CONTROL ON ALL APPROACHES

When stop signs were used on all approaches to the three intersections studied with this type of control, they were installed on the near side as well as the far side of the intersections, and advance warning signs were erected. On the one-way streets, signs were mounted on the left as well as the right-hand side of the street. Even under these conditions, however, it was necessary to discontinue the studies when the traffic volume reached the practical capacity of the intersections. The study had to be terminated after 13th Street had been operating one-way for little more than an hour and before the height of the afternoon rush was reached. The frequency with which a police officer was required to regulate the various movements immediately before the study was terminated made it apparent that the practical capacities of the intersections for this type of control were reached.

Table 1 shows traffic volumes for the heaviest hour and for the heaviest 10-minute period of the study with stop-signs on all intersection approaches. For each intersection, the volumes are shown separately for 13th Street and for the intersecting or cross street, and the combined total for the two is shown in the column, "Total for intersection." The average delay per vehicle during the period when the particular volume occurred is shown in Table 1. The peak volume on 13th Street did not always occur during the same time period as that for the cross street, and for this reason each intersection is listed twice in the table. The volumes for the cross streets as shown in the upper one-half of the table are those observed during the period when 13th Street was carrying its peak load. In the lower portion of the table the volumes shown for 13th Street are those observed when the cross streets were at a peak.

The highest 10-minute volume was observed at the intersection of 13th Street and Columbia Road, with 13th Street operating as a one-way street northbound and Columbia Road one-way westbound. The rate of flow on 13th Street during this period was 2,280 vehicles per hour, while the rate on Columbia Road during the same period was 762 vehicles per hour. The total volume for the intersection, 3,042 vehicles per hour, is equivalent to an average of 507 vehicles per lane per hour for all lanes entering the intersection. The accompanying delay averaged 31 seconds per vehicle to traffic on the cross street and 26 seconds per vehicle to traffic on the through street. This is very close to the delay of 30 seconds which was used as the criterion for practical capacity with stop-sign control on the cross streets. It seems reasonable to assume that most drivers would consider a delay greater than this intolerable at an all-way stop intersection just as they do at a cross street with stop signs.

During the 10-minute periods of maximum traffic flow listed in Table 1, it is noteworthy that the average volume per lane, including all approaches, was usually between 400 and 500 vehicles per hour. Most of the average delays to cross-street traffic observed during these same periods vary between 21 and 52 seconds. This indicates that the traffic load was between the practical and possible capacities of the intersections. It seems reasonable to assume that somewhat higher volumes might be carried with no increase in delay after a period of familiarization for the drivers. The practical capacity of these intersections with stop-sign control on all approaches and under the other existing conditions is therefore somewhere near 500 vehicles per hour for each lane used by traffic entering the intersection. This capacity is based on a reasonable traffic delay but does not take into consideration pedestrian delays or accident hazards which would probably tend to reduce the 500 figure to a somewhat lower value. The few pe-

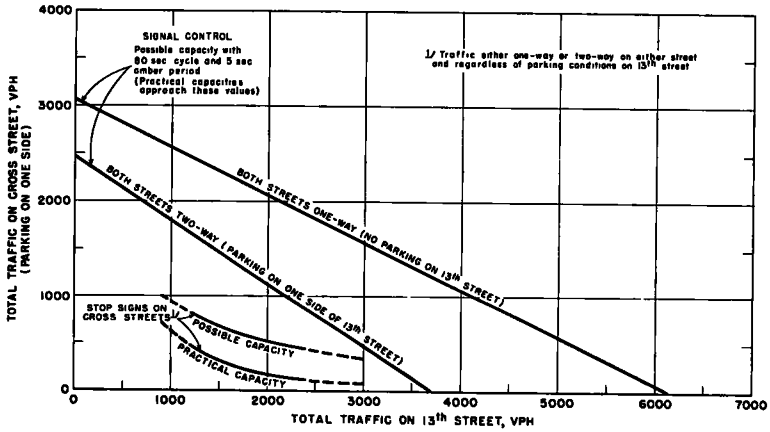


Figure 7. Capacities for two types of intersection control.

destrians at the intersections studied experienced long delays in crossing the streets even at moderate volumes.

Practical capacity for all-way stop control is compared with the capacities determined for the other types of control in Figure 8. This shows that under the traffic conditions prevailing at these intersections the capacity with all-way stop control is greater than with cross-street stop-sign control and less than with signal control. As previously stated, the capacities with stop signs on the cross streets are limited to those that permit a reasonable movement of the cross-street traffic.

At intersections where both streets are two-way, the practical capacity with four-way stop signs approaches the capacity with signals. The capacity with signals is, however, much greater than with stop signs on all approaches when both streets are one-way.

The delay to traffic was compared for the three types of control. At the traffic volumes observed, the delay to all traffic was found to be more with stop signs on all approaches than with signal control. Also, when the volume on the through street was below 1,800 vehicles per hour, stop signs on all approaches resulted in a greater average delay to all traffic than did stop signs on the cross streets. When the volume on the through street was greater than 1,800 vehicles per hour, the average delay to all traffic was less with stop signs on all approaches than with stop signs on the cross streets.

An analysis of the delays shown in Tables 6 and 1 indicates that the delay to cross-street traffic is far less when stop signs are used on all approaches than when used on the cross street only. An examination of other data obtained for lower traffic volumes shows this also to be the case for the lower volumes. The cross-street volumes in one direction during this study were never below 100 vehicles per hour. For cross-street volumes lower than this figure the relationship between delay and type of stop-sign control might be quite different from that found in this study.

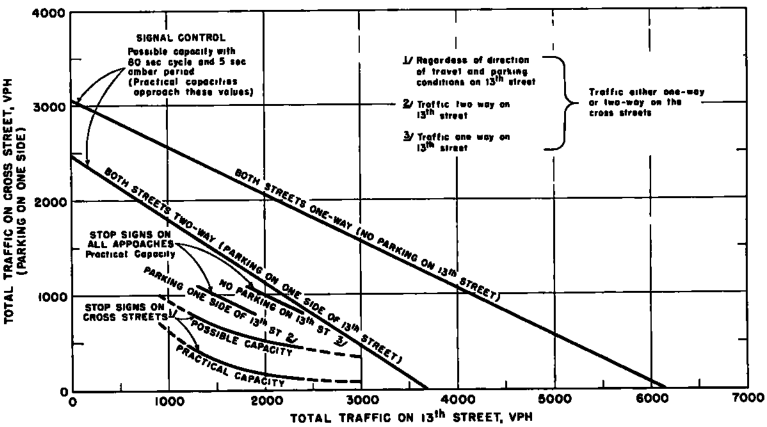


FIGURE 8 - CAPACITIES FOR THREE TYPES OF INTERSECTION CONTROL

Figure 8. Capacities for three types of intersection control.

SUMMARY OF FINDINGS

A definite value for either practical or possible capacity was determined for each of the three types of intersection control included in this study. The effect of one-way and two-way operation on the capacities of both through and cross streets and the effect of parking on the through street were also determined. These determinations have been based on the magnitude of the delays to traffic as well as on the traffic volumes which were observed. The studies were conducted at intersections where the through street was 40 feet wide and the cross streets 30 feet wide with parking on one side. The intersections were in a densely developed residential area with comparatively few pedestrians. Traffic on the through street was exceedingly heavy during the rush periods, and traffic on each of the four important cross streets was seldom less than 250 vehicles per hour during the day. The following are the more important findings for the prevailing roadway and traffic conditions:

Progressive Signal Control

1. The capacity of 13th Street with one-way operation and all parking prohibited was found to be 1,750 vehicles per hour of green per 10 feet of street width.
2. The capacity of 13th Street when operating two-way with parking on one side is 37 percent greater at two-way cross streets and 48 percent greater at one-way cross streets than with parking on both sides. With no parking on either side, the through-street capacity is 75 percent greater at two-way cross streets and 97 percent greater at one-way cross streets than when parking is permitted on both sides.
3. Changing from two-way to one-way operation on the through street, with no parking, increased the capacity of the through street 25 percent at intersections with two-way cross streets and 11 percent at intersections with one-way cross streets.
4. The capacity of the through street was increased 119 percent by the elimination of parking from both sides and the use of one-way rather than two-way operation.
5. Changing from two-way to one-way operation and eliminating parking on the through street increased the capacity of the two-way cross streets 11 percent and the one-way cross streets 20 percent.
6. The capacity of the one-way cross streets was 15 percent greater than the capacity of the two-way cross streets while there was two-way operation on 13th Street. The corresponding figure while 13th Street was operating as a one-way street was 24 percent.
7. Traffic delays with progressive signal control increased comparatively little with an increase in the traffic volume until the volumes approached very closely the possible capacities of the streets. This was true for delays to both through and cross-street traffic.

Stop-Sign Control on Cross Streets

8. The capacities of the cross streets with stop-sign control only on these streets were affected principally by the traffic volume on the through street. Whether the cross streets or the through street were operated as one-way or two-way streets made no apparent difference on the capacity of the cross street. The possible capacity of one-way and two-way cross streets decreased from 800 vehicles per hour to 400 vehicles per hour when the volume on the through street increased from 1,300 to 2,500 vehicles per hour. The decrease in capacity of the cross streets was not, however, directly proportional to the increase in volume on the through street.

9. Delay to traffic on the through street was practically nil, regardless of the traffic volume on the cross streets.

10. The delay to traffic on the cross streets was the principal criterion for a determination of possible and practical capacities with stop signs on the cross streets. The possible capacity is the volume which if exceeded even a slight amount will result in extremely long delays. For a given volume of traffic on the through street, the delay to traffic on the cross street when operating at possible capacity is more than double the delay when operating at practical capacity. Also, the practical capacity of a cross street is less than half of its possible capacity.

11. Changing either the through street or the cross street from two-way to one-way operation had no apparent effect on the delay to traffic on the cross street or the capacity of the cross street.

All-way Stop-sign Control

12. The practical capacity of the intersections with all-way stop-sign control was found to approach 500 vehicles per lane per hour including the lanes used by traffic on both streets. The validity of this finding under other conditions with a high traffic volume per lane on one of the streets and a low volume per lane on the other street was not determined.

13. With similar traffic volumes per lane on both streets, the delay per vehicle on the through street was approximately the same as the delay per vehicle on the cross street.

CONCLUSIONS

The following conclusions appear to be justified for the conditions under which these studies were conducted:

1. With properly coordinated progressive signal control, the practical capacity of the one-way street closely approaches its possible capacity. The delays to traffic when volumes are near possible capacity are not excessive under these conditions and only slightly greater than the delays at much lower volumes.

2. Intersection capacities are greater with progressive signal control than with either type of stop-sign control. When the through street is one-way, the intersection capacities with progressive signal control are substantially greater than with either type of stop-sign control. The average delay to all traffic is less with progressive signal control than with either type of stop-sign control, except possibly with stop-sign control at cross streets carrying exceedingly low volumes while the through street is carrying a high volume.

3. Possible and practical capacities for cross-street stop-sign control are much lower than with progressive signal control or all-way stop-sign control. Somewhat higher volumes than those found during this study might be practical with stop-sign control on the cross street when the intersection is located between signal-controlled intersections on the through street.

4. The capacity of an intersection at which all traffic is controlled by stop signs approaches that of an intersection with progressive signal control when both streets carry two-way traffic. When one or both of the streets carry traffic in only one direction, the capacity with all-way stop signs is considerably lower than that possible with progressive signal control.

5. The capacity of an intersection will vary greatly with the control and regulation of traffic. Additional studies are necessary to determine the most effective type of control for the many conditions that exist other than those included in this study.

In the application of the above conclusions, it must be remembered that they apply only to conditions similar to those found on 13th Street. The very high values for capacity with signal control cannot be used when estimating capacities because for most streets the results would be erroneously high. The very efficient operation of the progressive signal system should be reemphasized since it expedites the movement of exceedingly high volumes of traffic to a degree seldom equaled on streets of this type. It may be significant that most of the drivers on 13th Street during the peak period use this street daily and are therefore practiced in maintaining optimum speed and spacing.

These facts are important considerations when comparing the results of this study with those of other studies[3] which do not show the same advantages for signal control as compared with stop-sign control. There is close agreement, however, in the results of this study and those of other studies in the traffic volumes accommodated by the two types of stop-sign control. The fact that 13th Street can accommodate such high traffic volumes smoothly and at a reasonable speed for urban conditions is a tribute to the traffic engineers who operate the progressive signal system.

Discussion

HERBERT S. LEVINSON, Wilbur Smith and Associates, New Haven, Connecticut—French clearly indicates that the way to move traffic is to develop efficient usage of curb lanes through "all rolling" regulations, coupled with one-way movements and progressive signal timing.

Peak traffic movements on 13th Street in Washington approximate 700 vehicles per lane per hour. This is in accord with loadings observed on key radials in other cities.

French states that progressive timing develops practical capacities almost as high as saturation flow values. In this regard it is significant to note that the 80 percent relationship between the two capacity criteria can be demonstrated, assuming random arrivals. Platoon movements, with gaps between successive platoons, have, if at all, a very limited and particularistic randomness. Practical capacity in a progressive signal system is closely related to the capacity in vehicles per hour of the "through bands." This "band capacity" is generally high on a progressively timed one-way street.

French's analysis of vehicular operations at stop sign controlled intersections should also indicate the effects of the location of cross intersections as related to the time-space pattern of the through street. For example, midway between two signals that operate essentially simultaneously, gaps are generally well defined, even with heavy loadings. On the other hand, midway between two signals, ½ cycle offset from each other, there will be very few breaks in traffic, even under moderate volumes.

ALEXANDER FRENCH, Closure—It is true, as Levinson points out in his comments, that the high traffic volumes observed during this study are not unique. Since sufficient capacity to carry such volumes is not easily obtained on most city streets, many locations where high volumes are carried must be studied to determine the principal factors which influence the traffic-carrying capacities of streets. The Department of Traffic and Operations of the Highway Research Board is now engaged in just such a study. The Highway Capacity Committee of this department is cooperating with traffic engineers in cities throughout the nation to obtain traffic data for a large number of high-volume intersections. A result of this study will be a more complete understanding of the traffic-

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[3] "A Capacity Relationship Between Four-Way Stop Intersection Control and Fixed-Time Traffic Signal," by James Madison Hunnicutt, Jr. Thesis, Bureau of Highway Traffic, Yale University, 1954.

"A Comparison of Delay to Vehicles Crossing Urban Intersections, Four-Way-Stop versus Semi-Traffic-Actuated Signal Control," by Edward M. Hall. Student Research Report No. 4, The Institute of Transportation and Traffic Engineering, University of California, January 1952.

carrying capacity of intersections. Until more information is available, the high capacities, such as those found in this study, will not easily be duplicated. There is, therefore, as yet no assurance that such high capacities can be designed into a city street.

A comparison of delays to cross-street traffic shown in Table 5 of the report supports Levinson's belief that signals at intersections on the through street will affect the operation of intermediate cross-street approaches that are controlled by stop signs. With 13th Street operating one way, for example, the delay to cross traffic at Columbia Road is considerably greater than at Harvard Street. Since Harvard Street is closer to Euclid Street, where the signal was operating, it appears that the bunched platoon gradually spread out as it progressed up 13th Street, blocking the cross streets farther from the signal for a longer period than those near the signal. The significance of the relationship between vehicle spacing on the through street and the operation of the cross streets when controlled by stop signs could not be determined since so many other factors evidently had some effect. Data from studies at a large number of different locations must be analyzed to determine the significance of the many variables which influence the operating efficiency of street intersections. It is for this reason that the work now being carried on by the Highway Capacity Committee is of special value.

Starting Delay and Time Spacing of Vehicles Entering Signalized Intersection

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Institute of Transportation and Traffic Engineering,
University of California at Los Angeles

● TWO parameters of traffic performance can be used to represent several important characteristics of intersection operation. The two parameters, investigated in this study, are starting delay and time spacing of vehicles entering a signalized intersection. Information on the variability of these parameters at an intersection and among intersections may have useful applications in studies of intersection capacity and signal timing.

Starting delay, designated D, is defined as the time in seconds required for the first vehicle to enter the intersection after the display of the green signal. This corresponds to the "entrance time for the first-in-line vehicle" used by Greenshields, Schapiro, and Ericksen (1).

Time spacing, designated S, is the average time headway in seconds between successive vehicles in an entering platoon. In the measurement of S the number of lanes in which traffic is moving is disregarded, and the entire intersection approach is considered a unit. Time spacing as used here agrees with the definition presented in the "Traffic Engineering Handbook" (2) in its discussion of traffic signal timing formulas. S applies only to platoon movement and is computed by dividing total time in a signal cycle used for platoon movement by one less than the number of cars in the platoon.

INTERSECTION CAPACITY

Maximum capacities for signalized intersections can be expressed in terms of these parameters S, time spacing, and D, starting delay. For each cycle the time used for actual traffic movement is nS where n is the number of vehicles entering and S is time spacing as defined above. This assumes that a time S follows the entrance of the last vehicle in each cycle. Since an allowance must be made for a platoon of vehicles to start, a time D must be added to each cycle. The total time t required for n cars to enter an intersection is then $t = D + nS$.

This equation may be written $n = \frac{t - D}{S}$, and if the green and amber time is considered available for movement, this becomes $n = \frac{g + a - D}{S}$ for the number which can enter in one cycle with green time g seconds and amber time a seconds. Use of green plus amber appears appropriate here since the formula allows S seconds to follow the last car before the green begins on the opposing phase.

To find the total volume N which can enter during an hour, n is multiplied by the number of cycles per hour, m.

$$N = \frac{m\,(g + a - D)}{S} \tag{I}$$

Equation (I) expresses maximum hourly capacity under fixed time control. The general expression for hourly capacity, which is applicable to any kind of signal control, uses G and A as total hourly green and amber time, thus:

$$N = \frac{G + A - mD}{S} \tag{II}$$

In the "Highway Capacity Manual" (3) basic capacity is established as 1500 vehicles per 12-ft. lane per hour of green or 1250 per 10-ft. lane per hour of green. Basic capacity assumes a continuous stream of cars entering the intersection at about 2.4-sec. intervals in 12-ft. lanes and 2.9 sec. intervals in 10-ft. lanes for the entire period considered and in this way uses a factor closely related to the S used here. Basic capacity does not allow for traffic delays. Neither does it allow for the effect of parked cars, turning movements, trucks and buses, or other traffic conditions, nor for normal fluctuations in traffic volumes.

The formulas presented previously are based on values of S and D applicable to an individual intersection approach and, when applied to that approach, take into account the effect of all traffic conditions peculiar to that intersection. No allowance is made for fluctuations in volume. In the formulas presented here it is assumed that all available time in every cycle will be used for movement.

Greenshields et al (1) have approached the capacity problem in terms of the capacity of a traffic lane. They report that the first car enters the intersection 3.8 sec. after the beginning of the green (a value corresponding to D as used here) and that successive cars enter at 3.1, 2.7, 2.4, and 2.2-sec. intervals until the sixth-in-line and all following cars enter at 2.1-sec. headways. Use of these headway figures permits computation of the number of cars which can enter in one lane during a given green signal period and, from this, hourly capacity per lane.

Greenshields' method for computing capacity is similar in many ways to that suggested in this paper. It differs in that it is based on single-lane capacity rather than that of the entire intersection approach as used in this paper. In addition, Greenshields' method utilizes, in effect, single values of S and D and suggests no method for adjusting headways derived from them to conditions at other intersections. One of the purposes of this paper is to determine whether S and D are constants for all intersections.

TRAFFIC SIGNAL TIMING

Several formulas have been proposed for optimum timing of traffic signals. The formula used by Earl J. Reeder in reference (2) above utilizes S as defined here, V, the average velocity at the intersection, and q*, the number of vehicles arriving at the intersection in fifteen minutes, for computation of cycle length.

The National Safety Council (4) and the Institute of Transportation and Traffic Engineering (5) have published another formula, similar in some respects. The NSC-ITTE formula is:

$$T = \frac{y_1 + y_2 + D_1 + D_2 - S_1 - S_2}{1 - .0011\,(q_1S_1 + q_2S_2)} \quad \text{*} \qquad \text{(III)}$$

where quantities are defined as follows:

T = cycle length in seconds
y = amber clearance period (sec.)
D = starting delay (sec.) as previously defined
S = time spacing (sec.) as previously defined
q = number of vehicles entering in 15 minutes*

The subscripts apply to values for the opposing signal phases, and data are used for the heavier approach on each phase.

The signal timing formula is extremely sensitive to small changes in the value of S and somewhat less sensitive to changes in D. This sensitivity of T to changes in S and D indicates that formula (III) will be of relatively little use if either quantity is extremely variable from cycle to cycle or if their values do not remain reasonably constant from day to day. The cycle length T computed from formula (III) is usually somewhat shorter than most traffic engineers prefer to use in practice.

The above discussion indicate that these parameters, S and D, may have useful applications to traffic problems. The usefulness of these parameters, however, appears to be dependent on the variability of S and D values—both at any one intersection and among different intersections and intersection approaches.

INVESTIGATION OF S AND D

This experiment was intended to measure some typical values of S and D, to examine the variability of these parameters both from intersection to intersection and from day to day at the same intersection, and, if possible, to relate these values and their variabilities to physical and traffic conditions.

Starting delays and time spacings were observed and the results analyzed at thirteen

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* n is used instead of q in the references cited. The notation has been changed in order to be consistent in this paper.

heavily travelled intersection approaches in the Los Angeles area. The following conditions prevailed at these intersections:

1. All intersections were signalized. Eight approaches were controlled by fixed-time and five by full traffic-actuated signals.
2. Data were collected only on heavily loaded approaches. Each platoon of vehicles entering the intersection from the studied approach during the period of observation started from a stop and usually contained at least ten vehicles.
3. There were no streetcars and very few buses on the approaches studied.
4. The intersection approaches studied carried at least two lanes of travel in each direction. The range of street widths studied was 50 ft. to 76 ft.

Observations were made between 4:15 and 5:45 p.m. on warm, dry days in the spring and summer of 1952, and, where possible, observations at a given intersection were made on five consecutive week days. Data were taken by five different individuals during the course of the study. The same observer took all data for a given intersection approach.

During each study period the observer recorded data for thirty-one signal cycles. Starting delay, D, was recorded as the time from the first display of green to the entrance of the first vehicle into the intersection. A vehicle was considered to have entered when its rear wheels crossed the pedestrian crosswalk line nearer the center of the intersection. The first vehicle could enter from any lane. Negative values for starting delay are possible, but at the intersections studied cross traffic was always heavy enough during periods when data were being taken to prevent cars from entering before receiving the green signal.

The time for platoon movement was recorded as the time from the entrance of the first vehicle into the intersection until the entrance of the last car of the platoon. The observer also recorded the number of vehicles entering during this time. Average time spacing, S, for a given cycle was determined by dividing the time for platoon movement by one less than the number of vehicles entering during that time.

If a cycle occurred in which at the beginning of the green the front rank of the approach was not fully occupied by stopped vehicles, data were not taken for that cycle. Under prevailing traffic conditions, however, a large reservoir of waiting vehicles were usually assured during the study periods. All data were collected during peak traffic periods.

Determining the end of a platoon was a judgment on the part of the observer. Observers were instructed to consider a platoon ended whenever any one lane was empty or whenever traffic entered the intersection without being restricted in any way by cars immediately ahead. Observers were urged, if necessary, to cut off platoons early in order to be certain that all cars counted were actually traveling in platoons. Data were not recorded for individual lanes; data were based on all cars entering from all lanes in one direction.

Possible differences in results among observers were studied. Three observers independently recorded data for time spacing for the same intersection approach. An analysis of variance [1] was made among the three individuals' data and differences among observers were found not significant at this particular intersection.

The observed data described above permitted computation of mean values and standard deviations for D, the starting delay, and S, the average time spacing between vehicles. A tabulation of physical and traffic characteristics for the thirteen intersection approaches is presented in Table 1 and a summary of the values for starting delay and time spacing is presented in Table 2.

Values of both parameters, S and D, were assumed to be normally distributed. The chi-square test of goodness of fit was applied to three intersections for each parameter, and for each parameter the hypothesis of normality was rejected of the .05 level for one of three arbitrarily selected intersection approaches. Principal departures from normal distribution are noted in a greater number of relatively large S and D values under actual conditions than would be expected in a normal distribution.

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[1] For this and other statistical techniques, Dixon and Massey (6) was used as the principal reference.

TABLE 1
PHYSICAL AND TRAFFIC CONDITIONS AT THIRTEEN INTERSECTION APPROACHES STUDIED

Intersection	Direction	Widths Study Street	Widths Cross Street	Estimated ADT[a] Study Street	Estimated ADT[a] Cross Street	Parking	Prohibited Turns	District
Beverly at Fairfax	West	57	60	14.5[b]	21.5[c]	None	Left	Business
LaCienega at Pico	South	71	69	6.5[c]	26.0[b]	Yes	None	Business
LaCienega at Third	North	70	56		18.5[b]	Yes	None	Lt Bus
Melrose at LaBrea	West	57	70	13.5[b]		Yes	None	Lt Bus.
Santa Monica at Beverly Dr.	West	60	60	17.5[b]	12.0[b]	None	None	Residential
Sepulveda at Olympic	North	60	86	14.5[b]	27.0[b]	None	None	Intermediate
Sepulveda at Slauson	South	62		17.5[b]	11.0[b]	None	None	Rural
Sepulveda at Sunset	North	56	50	13.0[b]	17.0[b]	None	None	Open
Sepulveda at Sunset	South	56	50	15.5[b]	17.0[b]	None	None	Open
Sunset at Sepulveda	East	50	56	9.5[b]	28.5[b]	None	None	Open
Sunset at Sepulveda	West	50	56	7.5[b]	28.5[b]	None	None	Open
Westwood at Pico	South	50	60	8.0[b]	24.5[b]	Yes	None	Lt. Bus
Wilshire at Sepulveda	West	76	52	18.0[b]	27.0[b]	Yes	None	Vets. Home

[a] Average Daily Traffic is given in thousands for one entering approach for study street and for total cross street traffic in both directions. Sources are as indicated. Six-hour counts have been multiplied by 2.5. Peak counts have been multiplied by 10.
[b] Division of Highways
[c] City of Los Angeles

STARTING DELAY D

The mean starting delays at the thirteen intersection approaches studied ranged from 2.91 sec. on Sepulveda at Slauson to 4.40 sec. on La Cienega at Pico. The effect of location on mean starting delay was tested for significance by analysis of variance. The hypothesis tested was that all thirteen mean starting delays were equal. The analysis is presented in Table 3. The hypothesis of equal means can be rejected at the .005 level of significance, and the effect of location is thus found to be significant.

Statistical tests were applied to the effect of day of data-taking on the mean value of D for each intersection approach. Where variances of D on the different days were homogeneous, the analysis of variance was used; where variances were not homogenous, a modified t-test was used. The mean value of D for any one of the five days was significantly different from the five-day mean on only one of the thirteen intersection approaches studied (Sepulveda N at Olympic). At the twelve other intersections the effect of day of data-taking on D was not significant at the 5 percent significance level.

Reference to Table 2 shows that the highest five-day-average standard deviation, 1.52 sec., was observed on Sepulveda southbound at Sunset while the lowest five-day-average standard deviation, 1.04 sec., was on Sepulveda northbound at Sunset. The extreme values of variability were found on opposite approaches to the same intersection. Except for the four approaches to the Sunset-Sepulveda intersection, low mean starting delay is associated with low mean standard deviation.

The lowest standard deviation is 26 percent of the mean and the highest 39 percent. Seven of the 13 intersections have std. dev./mean values between 30 and 33 percent.

The mean value for starting delay, D, for the thirteen approaches studied is 3.83 sec. with an average standard deviation of 1.27 sec. within each day. The average standard deviation among five means for different days is 0.27 sec.

Comparison of mean delay values with various intersection characteristics does not reveal any one factor which appears to have a consistently important effect in increasing or decreasing the value of D. Volume of vehicle and pedestrian cross traffic, gradient of the approach, and the extent of right turning movements may be important factors in determining starting delay. Width of the pedestrian crosswalk may have some effect for very wide and very narrow widths. There are insufficient data in this study to permit careful evaluation of the effects of all variables.

Examination of the data indicates that starting delay is independent of street width and the type of signal installation at the intersection. The visibility of signal faces may be a factor, but this could not be evaluated from these data.

TIME SPACING S

Average time spacing for the approaches studied ranges from 0.95 to 1.63 sec., and mean values are significantly different among different approaches studied. The hypothesis

TABLE 2
OBSERVED STARTING DELAY AND TIME SPACING VALUES

Intersection	Dir.	Day	Starting Delay Mean	Starting Delay Std. Div	Time Spacing Mean	Time Spacing Std. Div	5-Day Averages D Mean	5-Day Averages D Std Div.	5-Day Averages S Mean	5-Day Averages S Std. Div.	Grand Std. Dev. D	Grand Std. Dev. S
Streets with Parking												
LaCienega at Pico	(71')S	M	4.14	1.30	1.27	0.26	4.40	1.38	1.15	0.23	1.377	0.237
		T	4.56	1.21	1.12	0.25						
		W	4.38	1.42	1.15	0.23						
		Th	4.49	1.48	1.09	0.21						
		F	4.45	1.50	1.10	0.20						
LaCienega at Third	(70')N	M	3.41	1.29	1.13	0.21	3.87	1.28	1.14	0.22	1.298	0.217
		M	3.13	1.39	1.16	0.23						
		T	3.97	1.26	1.10	0.17						
		T	3.60	1.27	1.13	0.23						
		W	3.49	1.21	1.16	0.26						
Melrose at LaBrea	(57')W	M	3.71	1.17	1.53	0.25	3.96	1.40	1.53	0.26	1.439	0.252
		T	4.32	1.03	1.57	0.27						
		W	3.91	1.67	1.53	0.26						
		Th	4.27	1.67	1.52	0.26						
		F	3.60	1.45	1.49	0.24						
Westwood at Pico	(50')S	T	3.51	0.94	1.58	0.27	3.69	1.13	1.63	0.32	1.255	0.306
		W	3.65	1.14	1.62	0.34						
		Th	3.70	1.31	1.61	0.32						
		F	3.71	1.01	1.66	0.29						
		F	3.89	1.24	1.68	0.31						
Wilshire at Sepulveda	(76')W	M	3.74	0.94	0.94	0.17	3.77	1.19	0.95	0.14	1.266	0.166
		T	4.52	1.34	0.99	0.11						
		W	3.19	0.96	0.95	0.18						
		Th	3.56	1.27	0.98	0.10						
		F	3.85	1.43	0.91	0.16						
Streets with No Parking												
Beverly at Fairfax	(57')W	M	3.56	1.26	1.23	0.19	3.79	1.23	1.26	0.24	1.247	0.236
		T	3.69	1.29	1.28	0.23						
		W	4.21	1.55	1.30	0.35						
		Th	3.43	1.12	1.26	0.17						
		F	4.08	0.91	1.25	0.18						
Santa Monica at Beverly Dr	(60')W	T	2.94	0.94	1.21	0.17	3.22	1.16	1.29	0.20	1.242	0.210
		W	3.39	1.30	1.34	0.21						
		Th	3.13	1.13	1.27	0.20						
		F	3.24	1.11	1.34	0.24						
		F	3.38	1.31	1.30	0.20						
Sepulveda at Olympic	(60')N	M	4.08	1.56	1.30	0.28	4.35	1.34	1.29	0.25	1.485	0.257
		M	4.42	1.30	1.36	0.24						
		T	4.90	1.59	1.22	0.24						
		T	3.73	0.95	1.20	0.27						
		Th	4.42	1.30	1.38	0.23						
Sepulveda at Slauson	(62')S	T	2.74	0.92	1.30	0.23	2.91	1.09	1.27	0.23	1.093	0.229
		T	2.99	0.90	1.26	0.24						
		W	2.86	1.29	1.24	0.19						
		Th	2.94	1.33	1.30	0.25						
		F	3.01	1.00	1.23	0.25						
Sepulveda at Sunset	(56')N	M	4.26	1.27	1.28	0.15	4.01	1.04	1.31	0.17	1.050	0.171
		T	3.70	1.05	1.34	0.25						
		W	4.02	0.94	1.30	0.16						
		Th	4.09	0.90	1.29	0.14						
		F	3.97	1.05	1.34	0.13						
Sepulveda at Sunset	(56')S	M	3.97	1.74	1.41	0.25	3.92	1.52	1.36	0.24	1.532	0.249
		T	4.29	1.69	1.31	0.21						
		W	4.04	1.59	1.27	0.22						
		Th	3.72	1.26	1.40	0.27						
		F	3.60	1.33	1.40	0.28						
Sunset at Sepulveda	(50')E	M	4.04	1.32	1.71	0.32	4.20	1.27	1.63	0.24	1.255	0.249
		T	4.14	1.10	1.53	0.18						
		W	4.29	1.42	1.66	0.22						
		Th	4.56	1.14	1.66	0.23						
		F	3.99	1.35	1.57	0.25						
Sunset at Sepulveda	(50')W	M	3.45	0.87	1.36	0.29	3.63	1.42	1.43	0.25	1.340	0.263
		T	3.75	1.35	1.34	0.27						
		W	3.74	1.74	1.53	0.28						
		Th	3.29	1.19	1.38	0.18						
		F	3.93	1.95	1.52	0.24						

TABLE 3
SUMMARY OF ANALYSIS OF VARIANCE TEST OF EQUALITY OF MEAN D'S FOR ALL INTERSECTIONS

$H_O: \mu_1 = \mu_2 \ldots = \mu_{13} \quad \alpha = .005$

	Sum of Squares	d. o. f.[a]	Mean Square	F-Ratio
Among means	389.82	12	32.48	$F = \frac{32.48}{1.68} = 18.3$
Within groups	3353.78	1998	1.68	

For $\alpha = .005$, $F_{.995}(12, 1998) = 2.36$. Reject H_O if F greater than 2.36. Therefore, H_O may be rejected at $\alpha = .005$. Mean starting delay is significantly different for different intersection approaches.

[a] A total of 2011 individual valid observations were made. Four cycles of data were omitted.

of independence of S values taken on different days was tested, and there was significant difference among days at three of the 13 intersection approaches (La Cienega-Pico, Sepulveda-Olympic, Sunset West at Sepulveda). As in the case of D values, analysis of variance was used where variances were homogeneous and a modified t-test where they were not.

The standard deviations of S for the 13 approaches fall in the range between 0.16 and 0.31 sec. The mean standard deviation is 0.234 sec., and eight of the 13 fall between 0.225 and 0.265 sec.

The standard deviations of five daily means ranged from .03 sec. for Sepulveda at Olympic to .09 sec. on Sunset W. at Sepulveda. The hypothesis of equal mean values for the five days was accepted for all approaches with standard deviations of S less than .07 among days.

The data show that S is a function of intersection characteristics and that significant differences in time spacing values exist among different intersection approaches. Examination of the data indicates that the two factors having the greatest effect on time spacing S for the intersection approaches studied are (1) street width and (2) parking conditions. The "Highway Capacity Manual" (3) reports studies which suggest that intersection capacity is increased almost linearly by additional feet of width and that capacity is a function of street width rather than simply of number of lanes. The studies conducted here appear to support these findings.

For the purpose of analysis the intersections studied were divided into two groups depending on whether or not parking was present on the approaches. Parking is permitted on La Cienega, Melrose, Westwood, and Wilshire. Parking is also permitted on Sepulveda at Slauson, but there was no parking here during any periods of data collection. Parking is prohibited at least during peak hours on the other streets studied.

Average time spacing S is plotted against street width in Figure 1. The data in squares are for streets with no parking, and Line A is the linear approximation for these data. The data in circles are for streets with parking permitted, and Line B is the linear regression line for these data. Both relationships were approximately linear within the range of street widths studied.

For the entire range of possible street widths, however, the relationships can be expected to be curves approaching the axes asymptotically, and Lines A and B are approximations of portions of these curves.

The regression lines for the assumed linear relationship of starting delay to street width are as follows:

Parking: $S = 2.98 - 0.026\ w$
No Parking: $S = 2.63 - 0.023\ w$

where w is the curb-to-curb street width.

Left turns were permitted at all intersections having no parking except on Beverly at

Fairfax. Beverly Boulevard is 57 feet wide here, and the S value falls below the assumed linear relationship of Line A.

Two instances of very good agreement between S values for intersection approaches of the same width are noted in the data. The two approaches studied on La Cienega have mean values for S of 1.14 and 1.15 sec. for widths of 71 and 70 ft., respectively. For streets without parking, Sepulveda at Olympic and Santa Monica at Beverly Drive are both 60 ft. wide, and the mean S value was 1.29 sec. for both approaches studied. On the other hand, mean values of S for eastbound and westbound approaches of Sunset Boulevard at Sepulveda are markedly different, apparently due to turning movements.

Within the range of street widths studied, smaller values of S were found for streets with no parking than for streets of the same curb-to-curb width but with parking. The difference between S values for the two parking conditions appears to decrease as the width increases. Better curves presumably can be drawn when data have been collected for more intersections and on a wider range of street widths.

CONCLUSIONS REGARDING S AND D

Several conclusions with respect to starting delay, D, can be drawn from examination of the data presented here.

1. There is a significant difference in starting delay among approaches at different intersections and among different approaches to the same intersection.

2. In general starting delay on one weekday can be considered equal to starting delay for all weekdays since no significant difference among mean D's on five different days was found at twelve of the thirteen intersections.

3. Factors which influence starting delay and are responsible for the difference in D among various locations have not been isolated and identified in this study. Volume of vehicle and pedestrian cross traffic, gradient, and percent of right turns are factors believed to have some effect. Type of signal control, width of the street, width of the cross street, and width of the pedestrian crosswalk appear to have little or no effect in the cases studied.

4. Starting delays are in most cases normally distributed. Departure from normality, where it exists, is in the form of positive skewing with a long tail of high values of D. In this study standard deviations of delays on a given day were found to be of the order of 1¼ sec., about 30 percent of the mean. A standard deviation of about ¼ sec. was found among mean values for five different days on the same intersection approach.

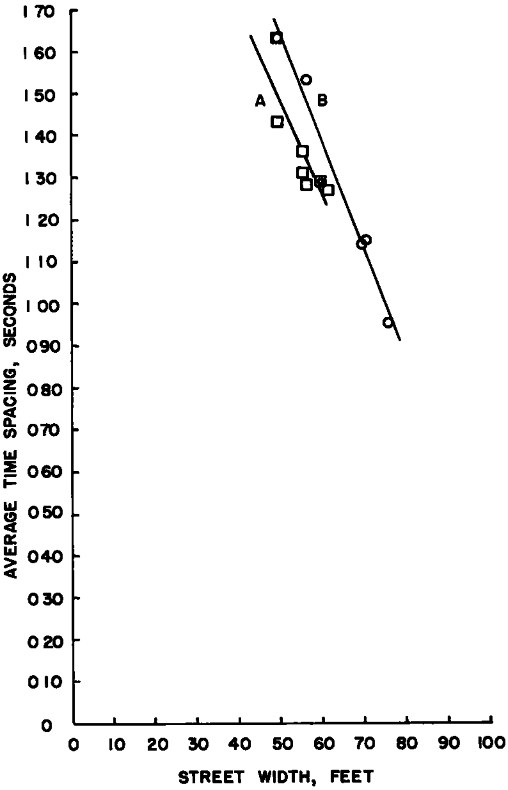

Figure 1. Time Spacing, S, as a function of street width. Curve A (points marked by squares): Approaches without parking. Curve B (points marked by circles): Approaches with parking.

Conclusions which may be drawn about time spacing, S, are the following;

1. Average time spacing is significantly different for different intersections, but the mean value obtained for one weekday will not usually differ significantly from that for other weekdays.

2. Although normality was not tested for all cases, time spacing appears generally to be normally distributed. The average standard deviation for an intersection approach is 0.23 sec., and this does not vary greatly from intersection to intersection. The dis-

tribution of mean S values for intersection approaches shows that all standard deviations of means among days are less than 0.1 sec. For six of the 13 approaches the standard deviation of daily means was less than .04 sec.

3. Time spacing on a given approach appears to be primarily a function of street width and parking conditions for the intersections studied. For streets with parking and without parking, time spacing S is approximately a linear function of street width over the range of street widths studied.

The above discussion of the experimental results indicates clearly that starting delay D and time spacing S are functions of conditions at individual intersections. Values for these parameters do not vary significantly from day to day in most cases. These statements are made for periods when traffic enters the intersection in platoon movement and there is a reservoir of waiting vehicles at the start of each green period, and are based on observations made only on week days.

These data indicate that while values of both D and S vary considerably from cycle to cycle, mean values vary somewhat less from day to day. This variability must be considered in applying these parameters to traffic problems and may restrict their usefulness

Additional studies of S and D for a wider range of intersection types are necessary to define in more detail the relationship of these parameters to physical and traffic conditions existing at intersections. A wider range of street widths should be studied. Turning movements should be studied for effect. No high volume downtown intersections were included among those studied here, but data for such locations are necessary for complete understanding of S and D.

APPLICATIONS TO CAPACITY

The maximum capacity of an intersection approach is related to D and S as discussed above and can be calculated using formula (II). Maximum capacity N can be estimated for a given intersection if S is measured for that approached or estimated from curves similar to those presented in this paper. S is a function primarily of street width and parking conditions, and a typical D may be assumed since N is not affected greatly by small changes in D.

Formula (II) shows that while intersection capacity can be increased by decreasing either S or D, a reduction in S will give the greater proportional increase in N. The results of this study indicate that S can be decreased by widening or by prohibiting parking. Only one intersection was studied at which left turns were prohibited, and the data, though inconclusive, indicate that time spacing was thus reduced. The "Highway Capacity Manual" (3) suggests several methods for increasing what it calls practical and possible capacity. Any of these operational measures except those which alter signal cycles can be effective if and only if they act to reduce either average time spacing of vehicles in platoon movement or the starting delay.

SIGNAL TIMING

Because of relatively large percentage variation in values from cycle to cycle, D and S are of limited value in determination of optimum signal timing. Formula (III), which is highly sensitive to changes in S and D, appears to require modification to allow for this variation if it is to accomplish the objective for which it was developed. To accommodate the traffic in 95 percent of all cycles in the peak period, Formula (III) might be improved by adding two standard deviations each to S and D in order to produce a larger value of T. The variability of S and D is so great, however, that their use in computing optimum cycle length is of doubtful value.

Use of these parameters is far more appropriate in determining the most effective cycle division when a total cycle length has been fixed by other considerations. The green time on any phase at a signalized intersection is composed of time for starting, time of actual vehicle movement, and extra time during which there is no movement. The last-named component should be divided between or among phases in proportion to the times actually required by each phase for the other components.

Expressed in terms of S and D, the time required for one phase per cycle is D + nS, where n is the number of vehicles entering that cycle. A reasonable basis for assigning a proportion of total green time to the two phases is:

$$g_1 : g_2 :: (D_1 + n_1S_1) : (D_2 + n_2S_2) \qquad \text{(IV)}$$

The above formula may be modified in one of several ways to meet individual conditions. Where the amber periods are used extensively for movement of vehicles, g can be replaced by (g + a). Where nS/D is very large, D can be omitted from consideration. Since extreme precision in cycle division is seldom required, the latter approximation can usually be used. This would divide the green time in direct proportion to the ratio of nS values for the heaviest legs of the opposing phases.

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Effect of Parked Vehicle on Traffic Capacity of Signalized Intersection

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● PARKING and parking restrictions are known definitely to affect the traffic capacity of intersections, but more information is needed on the extent of this effect and how it occurs. The research described in this paper was undertaken primarily to evaluate quantitatively the relationship of parking to intersection capacity for the case of a single car parked in the intersection approach.

Studies reported in the "Highway Capacity Manual" give typical values for capacities under various physical and traffic conditions, including parking, and factors for increasing or decreasing capacity as the result of changes in these conditions. These values and factors are based on reports of traffic volumes observed throughout the United States under these various conditions. The research described in this paper is based, however, on the detailed study of traffic performance on a single intersection approach under reasonably constant conditions except for parking a single car close to the intersection.

The two principal traffic capacity problems which were studied are these:

1. The restrictive effect of a single car parked in the intersection approach.
2. The effect on capacity of varying the distance of the parked car from the intersection

With respect fo the first of these problems, many traffic engineers apparently believe strongly in the importance of complete parking prohibition on heavily traveled streets. This has led to adoption of no-stopping "tow-away" regulations under which vehicles parked in violation during peak hours may be towed off the street and impounded. Emphasis of the tow-away appears to be on removing isolated parked cars along curbs which are otherwise clear. There are, however, few reported quantitative data on the effect on traffic capacity of such isolated parked cars.

The "Highway Capacity Manual" (1) contains two brief references to the effect of prohibiting parking near intersections. These are pertinent to the second problem mentioned above. A footnote under one section explains that these conclusions are based on rationalization of available facts and data but that there are insufficient data for statistical analysis. The manual states, with this qualifying footnote:

> "If parking is prohibited more than twenty feet in advance of the crosswalk, add P(D-20)5G percent, where P is the total percentage of turns, D the distance in feet from the crosswalk where parking is prohibited, and G the seconds of green per signal cycle. P cannot exceed 30, and D cannot exceed (5G-20)."(p. 89)

This formula suggests that capacity increases proportionally as P and D increase and as G decreases.

A later section on the same page and without the qualifying footnote states that prohibition of parking in advance of the intersection for a distance in feet equal to five times the green period in seconds is equivalent to a complete prohibition of parking as far as capacity is concerned.

The experiment described here is intended to evaluate critically the effect of a single car parked in the intersection approach. Parking and unparking operations presumably contribute to a reduction in capacity where parking is permitted but are not considered in this study. This study is limited to the effect on intersection capacity of a single car parked in the intersection approach. This absence of parking and unparking is an important difference between the single car situation and the line of parked cars found in a "parking permitted" situation. In addition, the increased maneuvering space made available beyond the parked car away from the intersection may contribute to the difference between the single car case and that of a line of parked cars.

THE EXPERIMENT

An empirical approach was used to evaluate quantitatively the effect of a single parked car in the intersection approach. Parking one car at a predetermined location, observing traffic behavior, and comparing data thus obtained with those obtained without parking should enable a careful experimenter to evaluate the restrictive effect, if any, of the parked car.

To examine the effect of a single parked car on intersection capacity, data were taken at an intersection approach operating under capacity conditions without parking and with a single car parked at two different distances from the intersection. Data were collected on the number of cars entering during short time intervals in each signal cycle as measured from the start of the green period. Mean volumes entering under different parking conditions in these time periods were compared and tested statistically for significant differences. Significant differences among parking conditions, if any, indicate whether or not the parked car and its location restrict intersection capacity and show the portions of the signal cycle in which any such restriction is produced.

Since data were necessarily collected on several different days, the effect of day of data-taking on entering volumes was tested for significance. If there is no significant effect, data taken on several different days under the same parking conditions may be pooled in order to increase sample sizes.

The effect of parking on lane volumes was tested using data for separate lanes and following methods similar to those described above for total volumes.

The analyses described above were designed to test for significant effects of parking on entering volumes under capacity conditions and for significant differences in volumes between parking distances. These analyses indicated the magnitude of these significant effects, if any, and the parts of the signal cycle and the traffic lanes in which they occurred.

The intersection selected for the experiment and the method of data collection and analysis are discussed in the sections which follow.

The southbound approach of Sepulveda Boulevard at Wilshire Boulevard near the Los Angeles campus of the University of California was selected as the site for this experiment. This intersection is on a state highway (Sepulveda) in unincorporated county territory, and traffic is under jurisdiction of the California Highway Patrol. Inspector R. R. Emmett of the Highway Patrol and Captain Walter Sequiera of the Patrol's West Los Angeles office extended their full assistance to aid the experiment.

Sepulveda Boulevard is 52 feet wide curb-to-curb at its intersection with Wilshire Boulevard. Parking is not specifically prohibited by posting on either approach. There is almost no parking at any time, however, since the highway runs through the grounds of a veterans' facility. A cemetery lies on one side of the street, and the other side is vacant near the street. Both sides are fenced.

Under normal operation two lanes of traffic move southbound along Sepulveda Boulevard to enter or cross Wilshire. Sepulveda Boulevard is 26 feet from center to the edge of the roadway, and one 11-foot lane is marked. Vehicles occasionally move two abreast in the remaining 15 feet of width near the intersection. A large number of the vehicles using the extreme right edge of the roadway make right turns onto Wilshire Boulevard. Parking a typical car on this southbound approach would reduce usable street width on that side of the center line to twenty feet. Two lanes of traffic could continue to pass this point although the outer (curb) lane would be only 9 feet wide instead of 15. The extreme right edge of the roadway could be used for movement only between the car and the intersection. A sketch of this intersection is presented in Figure 1.

On the southbound approach, therefore, parking a single car would not block a full traffic lane. Parking reduces the width of one lane and prevents use of the extreme outer portion of the roadway, used primarily for right turns in normal operation. Introduction of the parked car does not block a normally used lane; therefore, differences in traffic movement between the no-parking and parking conditions will be less sharply marked than presumably would be the case if the street were a few feet wider or narrower. The fact that an entire lane is not blocked presents some important advantages to the experimenter. The parked car is likely to cause less disturbance among motorists, and accidents appear to be less likely.

The intersection selected for study is controlled by a fixed-time three-light signal operating on a 60-second cycle. Each street, Wilshire and Sepulveda, is normally given 27 seconds of green and 3 seconds of amber. A California Highway Patrol officer operates the signal manually during periods of extremely heavy traffic. This is done primarily

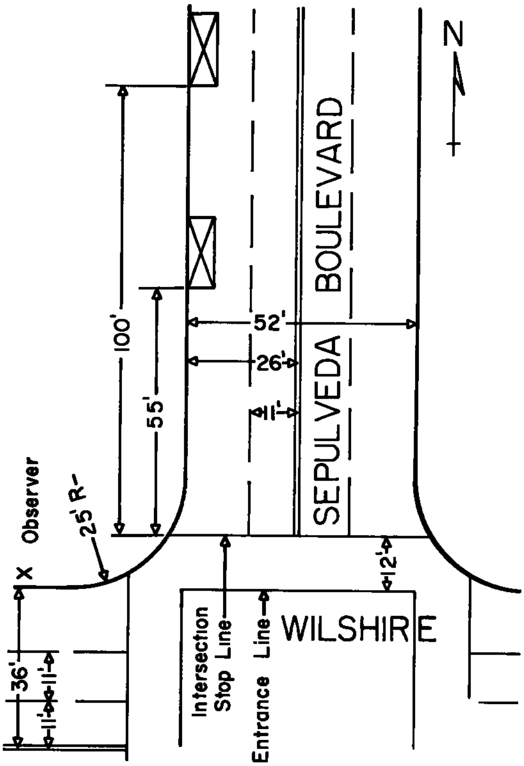

Figure 1. Sepulveda and Wilshire Blvds. showing positions at which cars parked in the experiment.

to avoid blocking an ambulance entrance on the nearby veterans' facility grounds.

To permit quantitative evaluation of the effect of the parked car, data were collected on the number of vehicles entering from each lane in various short time intervals within the 30-second green and amber period of the signal. The data collection method adopted utilized the Esterline-Angus 20-pen operations recorder. An observer recorded a pip on the record chart at the time each vehicle entered the intersection. Only one man was required to collect data, and the events recorded on the chart were so simple that few errors in recording or transcription could be made.

This method produced a graphic record showing the time of entrance of each vehicle from each lane and the beginning of the green signal indication.

Since there was no automatic recording of any kind, human error could be present in recording all events. One-half second on a given observation is estimated as the maximum error. To make a larger error on an entering vehicle would require that the button be pushed when the car was 15 to 20 feet from the stop line.

Throughout all data recording, a car was recorded as entering the intersection when its rear wheels crossed the pedestrian crosswalk line nearest the intersection center.

A brief study of entering volumes per cycle was made during weekday afternoon peak periods in order to determine the necessary sample size. Data for 200 cycles on these weekdays indicate that the mean volume entering the intersection per 30 seconds green period was 19.6 with a variance of about 22. In this study a difference of one vehicle per cycle among the no-parking and the 55-foot and 100-foot parking conditions should be detected. To detect a difference of 1.0 between means at the .05 level of significance with a probability of .80 would require 274 samples under each condition according to the method of Harris, Horvitz, and Mood (2).

To reduce the variance of the number of vehicles entering per cycle, data were collected on Sunday afternoons, when from approximately 3:30 to 6:30 p.m. the southbound approach is overloaded. There is a continuous reservoir of waiting vehicles during this period. Average volume entering per cycle is higher since there is traffic demand for all of every green signal period. Because this demand remains nearly constant during the entire period, variance of the entering volume is lower.

Twenty-five cycles were observed under each of two conditions on Sunday, May 3, 1953, and a pooled mean of 22.92 vehicles entering per cycle with a pooled variance of 6.24 was obtained.

Variance of 6.24 requires a sample of 81 in each condition to reject the hypothesis of equal means for a difference of 1.0 with $\alpha = .05$ and $\beta = .80$. The desirability of collecting data during the Sunday peak traffic period is readily apparent.

Data-taking periods were scheduled to obtain at least 81 cycles of data under each parking condition. All cycles were to be recorded under overloaded intersection conditions: a continuous reservoir of vehicles waiting to enter. The parked car was usually moved only once during an afternoon since moving it required that the experimenter leave his equipment unattended. On the basis of observations at the intersection and analyses of the data, time of day within the 3:30 to 6:30 p.m. period had no effect on the traffic.

TABLE 1

SCHEDULE OF DATA COLLECTION

Date	Number of Cycles Recorded: No Parking	Parking at 100 ft.	Parking at 55 ft.
May 3	25	25	—
May 10	30	40	30
May 24	31	32	—
June 21	—	—	70
TOTAL	86	97	100

Data for no parking and parking at 100 feet were collected on three Sundays in May 1953. At least 25 cycles were observed under each condition each day. Weather was clear on all of these days. Daylight saving time was in effect so that daylight continued for more than an hour after data taking was concluded.

Short periods were noted in which no cars were waiting to enter, and these were excluded from the data. Exceptional conditions such as a stalled car entering or cross traffic blocking the intersection also caused cycles to be excluded.

During the data taking on May 10 a car stalled in the intersection approach and was pushed to the curb 55 feet from the nearest crosswalk line. The car remained parked in this position for 30 signal cycles while waiting for repair. Data were taken continuously during this period to determine the effect of parking closer than 100 feet to the intersection. Thirty cycles were recorded on May 10, and 70 more cycles were recorded on June 21 with a car parked in this same position. Table 1 shows the days of data taking and the number of cycles recorded for each condition of parking.

The order of parking and no-parking conditions was varied so that neither condition was observed first on every day. Signal operation was not influenced by the experimenter. The Highway Patrol officer used manual operation when traffic backed up to Sawtelle on Wilshire.

As described above, the time of entrance of each vehicle into the intersection was recorded by lane and by turning movement, if any. Data were transcribed from the Esterline-Angus chart by numbering cars in a given cycle serially and recording each to the nearest half-second of its intersection entrance. The numbers of cars in 5, 10, 15, 20, 25, and 30 seconds were tabulated and summarized for each day and each condition.

ANALYSIS OF THE DATA

A cumulative total of vehicles entering under no-parking, 100-foot, and 55-foot parking conditions is presented in Table 2 and Figure 2. All data are presented in terms of volume per 100 cycles and show the cumulative totals by one-second intervals. For the full 30-second green period traffic entered without parking at the rate of 2356 vehicles /100 cycles compared with 2212 vehicles/100 cycles with a single car parked at 100 feet and 2215 vehicles/100 cycles at 55 feet. The three curves appear almost identical for about 15 seconds after the start of the green. The volumes entering in the first 13½ seconds are equal for no parking and 100 feet, 928 vehicles/100 cycles, and the no-parking cumulative volumes are greater than with parking continuously after this time. At 24 seconds the no-parking volume is a full second ahead of that with parking, 1849/100 cycles without parking compared to 1843/100 cycles for 25 seconds with 100-foot parking and 1835 for 55-foot parking. At 26½ seconds the no-parking volume exceeds that with parking by one vehicle per cycle. At 28 seconds the no-parking volume is 1½ seconds ahead of its counterparts. Throughout the cycle there is little difference between the curves for the two parking distances.

The effect of the parked car on volume entering this intersection is not noticed in the first 15 seconds of the green period, but parking allows less traffic to enter in a given time interval in the second half of the green period.

The sharp difference in volumes entering under the two conditions of parking and no-parking in the last 3 seconds cannot be attributed to the difference between manual and fixed-time operation of the signal. The 27-30 second interval is green under manual control but is amber under fixed-time operation. Forty-two of the 97 cycles (43 percent) observed with 100-foot parking had manual control, but only 25 of 86 no-parking cycles (29 percent) were manually operated. Thirty-one of 100 cycles with 55-foot parking were manually controlled. If an effect of the difference in signal operation is reflected in the 27-30 second interval, the cycles with parking would be expected to carry proportionally the greater volume.

Analysis of variance has been used to test the effect of parking and the effect of days on which data were taken. A three-by-two analysis of variance table was set up using data only for the first 25 cycles observed each day under each of two conditions: no parking and parking at 100 feet.

Volumes in the intervals 0-10, 0-15, 0-20, 0-25, 0-30 (full green period), 10-20, 10-25, 15-25, 15-30, and 20-30 seconds were analyzed. The values in each cell are assumed to be normally distributed. A rather large variance for no-parking data on May 3, first day of data taking, caused rejection of the hypothesis of equal variance for 0-25, 0-30, 15-25, and 15-30 at the one percent level. These intervals with non-homogeneous variance were not tested.

Table 3 shows the mean volumes entering in each of these intervals by days and by parking condition, and summarizes the six analyses of variance.

The effect of different days was found not significant for all of the intervals tested. There is no reason, therefore, to expect days to have a significant effect on the volumes in other intervals not tested this way. Parking was found to have a significant effect in three of the six intervals tested. No significant interaction between parking and days was found for any of the tested intervals.

TABLE 2
CUMULATIVE VOLUMES ENTERING BY ONE SECOND INTERVALS SEPULVEDA BOULEVARD SOUTHBOUND AT WILSHIRE BOULEVARD

May 3, 10, and 24, and June 21, 1953[a]

Seconds After Green Begins	Cumulative Volumes Entering: No Parking, 86-cycle Total	No Parking, 100-cycle Equivalent	Single Car Parked, 100 Feet, 97-cycle Total	Single Car Parked, 100 Feet, 100-cycle Equivalent	Single Car Parked, 55 Feet, 100-cycle Total
1	1	1	2	2	0
2	9	11	24	25	12
3	43	50	73	75	74
4	114	133	156	161	153
5	174	203	224	231	222
6	241	280	296	305	311
7	319	371	373	385	380
8	401	467	460	474	468
9	463	539	532	549	551
10	538	626	613	632	633
11	611	711	688	710	706
12	692	805	769	793	793
13	763	887	853	880	871
14	844	982	937	967	955
15	906	1054	1005	1037	1022
16	983	1155	1087	1121	1104
17	1057	1241	1169	1206	1178
18	1120	1311	1244	1283	1260
19	1196	1402	1335	1377	1349
20	1275	1483	1397	1440	1441
21	1357	1577	1483	1530	1523
22	1433	1665	1558	1607	1600
23	1503	1747	1645	1697	1679
24	1591	1849	1719	1773	1759
25	1662	1933	1788	1843	1835
26	1728	2009	1873	1932	1912
27	1803	2096	1942	2003	1997
28	1899	2207	2021	2085	2072
29	1963	2282	2095	2161	2150
30	2026	2356	2146	2212	2215

[a] All data taken on Sundays.

Since these six analyses show that the days of data taking had no effect on volumes entering the intersection, additional tests may be made for other intervals and additional data. While 86 cycles without parking and 97 with a car at 100 feet were recorded, only 75 of each could be used in the analysis of variance described above.

The assumption of independence of day of data-taking could be extended to data collecte with a single car parked at 55 feet. Therefore, comparisons among the three conditions, no parking and parking at 100 and 55 feet, could be made with the t-statistic and utilizing all data for each parking condition.

Tables 4 and 5 present the results of comparison among the three conditions as tested with the t-statistic. No significant differences in volumes between the 100-foot and 55-parking conditions were detected. When compared with the no-parking condition, tests involving the two parking distances produced almost identical results. Significantly more cars entered without parking than with parking at either distance for the first 25 seconds and for the full 30-second green (or green and amber) period. Significant differences in volume were noted in all the smaller intervals tested: 10-20, 10-25, 15-25, 15-30, and 20-30 seconds after the start of the green.

The volume without parking exceeds that with 100-foot parking by at least 0.81 vehicle per cycle with a probability of .95. No-parking volume exceeds that with 55-foot parking by 0.77 vehicles per cycle or more with the same probability.

These test results demonstrate clearly that parking has an effect and that the effect is noted almost entirely in the last half of the green period. In this case, significant differences were detected in the 10-20 second interval, but these were comparatively smalle than those found in later parts of the green period. The last ten seconds of the green

TABLE 3

MEAN VOLUMES ENTERING PER CYCLE IN SELECTED TIME INTERVALS WITHOUT PARKING AND WITH PARKING 100 FEET FROM THE INTERSECTION

Sepulveda Boulevard Southbound at Wilshire Boulevard
May 3, 10, and 24, 1953 (a)

Seconds After Green Begins	No Parking 3	No Parking 10	No Parking 24	No Parking Total	Parking at 100 Ft. 3	Parking at 100 Ft. 10	Parking at 100 Ft. 24	Parking at 100 Ft. Total	F-ratios Interaction	F-ratios Parking	F-ratios Days
0-10	6.32	6.27	6.19	6.26	6.52	6.08	6.47	6.32	0.87	0.00	0.16
0-15	10.80	10.43	10.42	10.54	10.48	10.13	10.56	10.37	0.77	0.30	0.09
0-20	15.36	14.57	14.65	14.83	14.48	14.20	14.56	14.40	0.27	3.72	0.83
0-25	19.72	19.20	19.13	19.33	18.60	18.08	18.69	18.43			
0-30	23.72	23.53	23.48	23.56	22.12	22.00	22.31	22.12			
10-20	9.04	8.30	8.45	8.57	7.96	8.08	8.09	8.08	2.60	4.88(b)	0.93
10-25	13.36	13.00	12.94	13.07	12.08	11.98	12.22	12.11	0.84	16.00(c)	0.43
15-25	8.92	8.77	8.71	8.79	8.12	7.95	8.13	8.07			
15-30	12.92	13.10	13.03	13.02	11.64	11.88	11.72	11.76			
20-30	8.36	8.97	8.81	8.73	7.64	7.80	7.72	7.72	0.46	20.90(c)	1.27

(a) All data taken on Sundays.
(b) Significant at .05
(c) Significant at .005

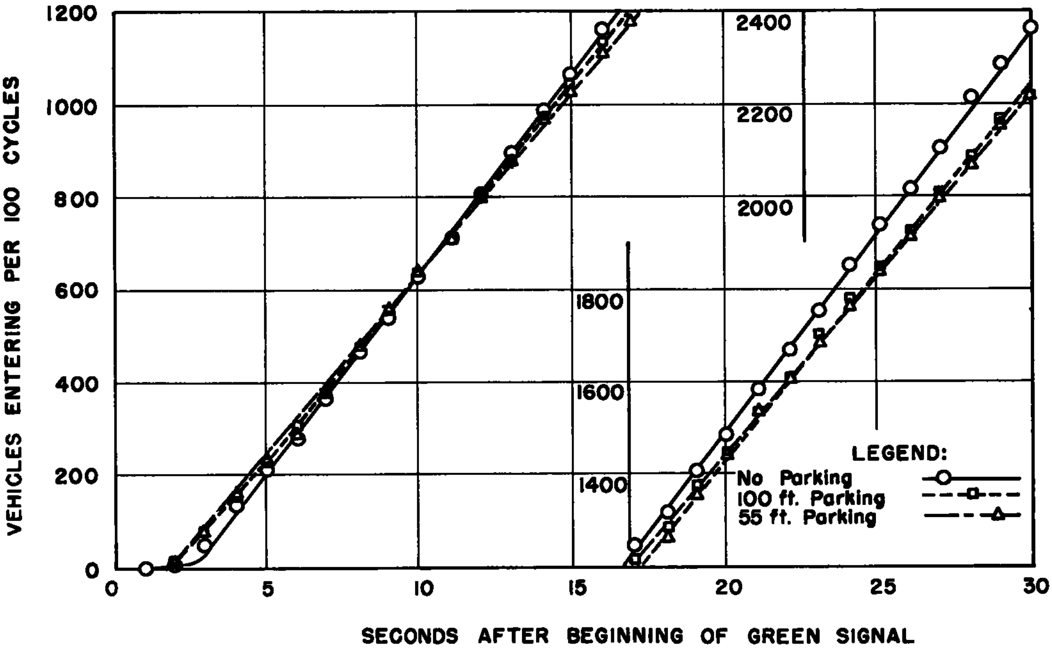

Figure 2. Cumulative entering volumes with and without parking.

demonstrated the effect of parking very clearly. For the entire green period the probability is .95 that the no-parking volume exceeds that with parking by at least 0.8 per cycle. In the last 10 seconds, however, the probability is .95 that no-parking volume is larger by about 0.6 per cycle.

Review of the physical situation involved suggests a more restrictive effect on traffic volume should result if the car is parked closer to the intersection. If this is so, additional

TABLE 4

MEAN VOLUMES ENTERING PER CYCLE IN SELECTED TIME INTERVALS WITH NO PARKING AND PARKING AT 100 FT. AND 55 FT.

Time Interval (Seconds)	No Parking	Parking at 100 Ft.	Parking at 55 Ft.
0-10	6.26	6.32	6.33
0-15	10.54	10.37	10.22
0-20	14.83	14.40	14.41
0-25	19.33	18.43	18.35
0-30	23.56	22.12	22.15
10-20	8.57	8.08	8.08
10-25	13.07	12.11	12.02
15-25	8.79	8.07	8.13
15-30	13.02	11.76	11.93
20-30	8.73	7.72	7.74

experimentation should result in significant volume differences between the 100-foot an 55-foot parking conditions. The data collected and analyzed here do not, however, support this contention.

Observed traffic volumes on this intersection approach were very much higher tha the theoretical capacities of the approach calculated according to methods of the "Highwa Capacity Manual." The approach is 52 feet wide, there are no left turns and 5 percent right turns, 10 percent of the traffic is commercial, and green constitutes 27/60 of the signal time. Following the method of page 79 of the manual, the possible capacity is 1088 vehicles per hour.

This computation assumes this intersection can be considered as "downtown with parking prohibited," which results in the highest capacity value. If it is classified instead as "intermediate," the possible capacity is reduced to 955 vehicles per hour.

Curves presented in the manual indicate that the capacity of a downtown intersection is reduced 29 percent by permission of parking and that in an intermediate area by 45 percen

The observed volumes for almost five hours of data taken averaged 1413 vehicles per hour with no parking and 1328 vehicles per hour with a car parked. The volume was reduced six percent by the parked car. The observed volume without parking averaged 30 percent higher than the calculated possible capacity under the most favorable conditions.

If the intersection is assumed to be part of a high-type facility, possible capacity is 1316 vehicles per hour.

As noted above, data for each condition of parking were collected under both fixed-time and manual control of the signal. For the purpose of this study manual operation differs from fixed-time only in that longer cycles were used under manual operation. The 27-30 second interval is amber under fixed-time but green under manual control. Green time in excess of 30 seconds in any cycle was not analyzed for effect of parked vehicles.

There is a probability of .95 that volume in the 25-30 second interval under fixed-time control is at least .13 vehicle per cycle less than that under manual control. This finding serves to reinforce the conclusions with respect to the effect of the parked car since more cycles with manual control were recorded with parking than without.

Having determined where in the green signal period the effect of a parked car becomes significant, lane volumes were studied. The analysis presented here is based only on the volumes per lane entering in the full 30-second green (or green and amber) period of each cycle.

A two-way analysis of variance was made to test the effects of day of data taking and of condition of parking. Analyses were made for Lane 1 (center), Lane 2 (curb), and right turn volumes. Normality was assumed, and the hypothesis of equal variance among the six cells was not rejected at the .01 level. Only the first twenty-five cycles for each cell were utilized in the analysis of variance.

Day of data taking was not significant for any of the volumes tested although the significant interaction (.05) in Lane 2 volumes suggests some effect due to days. The number of right turns under no-parking conditions was significantly greater than with a car parked at 100 feet.

Assuming that day of data taking has no significant effect on lane volumes permits comparison of volumes under the several parking conditions using the t-statistic. Significant interaction outlined in the paragraph above suggests a possible effect of days and may rais some question about the conclusions of the t-tests.

Lane 1, nearest the roadway center, was not significantly affected by parking. Mean volumes under the three conditions were nearly equal for this lane, and the t-test showed no significant difference.

Volumes straight through the intersection in Lane 2 were significantly greater without parking. This lane is also used by right-turners who have been excluded from this analysis. As in the case of earlier analyses using the full cycle and all lanes, no significant differences were detected between volumes with parking at 100 and at 55 feet.

The mean number of right turns per cycle varied greatly. Significantly more right turns were made without parking than with parking at 100 feet. The mean right turns with parking at 55 feet did not differ significantly from those under either of the other conditions. Mean right turns per cycle were 2.40, 1.67, and 2.03 for no parking, parking at 100 feet, and parking at 55 feet.

TABLE 5

TEST OF DIFFERENCE BETWEEN MEAN VOLUMES ENTERING WITHOUT PARKING AND WITH PARKING AT 100 FT. AND 55 FT.
t-STATISTIC APPLIED TO POOLED DATA

No parking: 86 cycles. 100-ft. parking: 97 cycles.
55-ft. parking: 100 cycles.

Time (sec)	No Parking vs. 100-Ft. Parking			No Parking vs. 55-Ft. Parking			100-Ft. vs. 55-Ft. Parking		
	d.f.	t	Diff.	d.f.	t	Diff.	d.f.	t	Diff.
0-10	181	0.63	NS	184	0.35	NS	195	0.27	NS
0-15	181	0.74	NS	161	1.42	NS	195	0.70	NS
0-20	181	1.74	NS	184	1.58	NS	195	0.08	NS
0-25	167	3.27**	0.37	184	3.31**	0.40	195	0.23	NS
0-30	181	4.60***	0.81	184	4.39***	0.77	195	0.07	NS
10-20	181	2.91**	0.17	164	2.83**	0.15	195	0.16	NS
10-25	156	4.80***	0.59	184	4.92***	0.64	195	0.33	NS
15-25	153	4.33***	0.40	184	3.63***	0.30	190	0.53	NS
15-30	181	5.83***	0.83	184	4.93***	0.65	195	0.86	NS
20-30	181	5.29***	0.63	184	5.06***	0.60	195	0.06	NS

* Difference significant at .05
** Difference significant at .01
*** Difference significant at .005
NS Difference not significant at .05

Column Headings:

d.f. Degrees of freedom
t t-statistic
Diff. Difference between means exceeded or equalled with probability of .95.
Where variances under the two conditions were not significantly different, the degrees of freedom are two less than the sum of the observations. Where variances are significantly different, the degrees of freedom are reduced. Refer to pp. 104-5, Dixon and Massey, Introduction to Statistical Analysis. (7)

The significantly larger number of right turns observed under the no-parking condition is difficult to explain and may affect some of the other results. There is insufficient evidence, however, to indicate whether or not parking had a significant causative effect on the mean number of right turns per cycle.

Additional analyses were made of the effect of parking on entering volumes for cycles with the same numbers of right turns. In general, the results of these tests with numbers of right turns equalized were the same as those for combined data.

As in the analyses presented above, the segregated analyses demonstrate that Lane 1 is not significantly affected by parking and that no real difference in any lane or in total volume exists as a function of distance parked from the intersection.

The effect of parking with turning conditions equalized on volumes in the interval 15 to

30 seconds after the beginning of the green was also studied. The mean values are presented in Table 6 and show that the mean number entering without parking in this interval is greater than with parking for any number of turns per cycle.

TABLE 6

MEAN VOLUMES ENTERING 15-30 SECONDS AFTER BEGINNING OF GREEN SIGNAL WITH NO PARKING AND PARKING AT 100 FEET AND 55 FEET

Cycles with Equal Numbers of Right Turns

Right Turns	No Parking	Parking at 100 Ft.	Parking at 55 Ft.
0	12.40	11.69	11.42
1	13.11	11.83	12.17
2	12.74	11.67	12.07
3	13.00	11.87	11.73
Over 3	13.48	12.00	11.91
Total	13.02	11.76	11.93

These results indicate that the greater number of right turns without parking does not affect the major conclusions of this paper. For cycles with the same number of right turns significantly larger volumes entered without parking than with a single car parked for most numbers of turns, and qualitative examination of the data suggests that significant differences would be found in all parking vs. no parking cases if a larger sample of cycles were analyzed.

SUMMARY AND CONCLUSIONS

At the overloaded intersection approach studied a single car was parked at the curb at distances of 100 feet and 55 feet from the intersection stop line. Data were collected on numbers and times of entrance of cars entering the intersection without parking and with the single car parked in each of these two positions. The analyses of these data constitute the body of this paper.

With the single car parked in the approach, traffic was able to pass it in two lanes. This is the same pattern as when there is no parking although the lane nearest the curb is considerably narrowed by the parked vehicle. Although other considerations governed the selection of this particular intersection for study, this chosen approach is typical of cases in which a complete new lane is not made available to traffic as a result of the parking prohibition. Effective width of the curb lane is increased, however, with resultant greater freedom of traffic movement.

All data analyzed as part of this experiment were collected under overloaded traffic conditions. There was a continuous reservoir of waiting vehicles on the approach under study, and volumes may thus be considered "capacity" volumes.

In these analyses the traffic pattern was not a function of the position of the parked car. With the quantity of data available, no significant differences were found between entering volumes with the car at 100 feet and at 55 feet from the intersection stop line, indicating that differences, if any exist, must be very small.

The parked car at either location had no effect on volumes entering the intersection during the first fifteen seconds of the green signal indication. From 15 to 30 seconds after the start of the green the effect of the parked car became increasingly pronounced. In the last 15 seconds the probability is .95 that the volume with no parking exceeds that with 55-foot parking by at least .65 vehicle per cycle and that with 100-foot parking by at least .83 vehicle per cycle. These differences represent between 5 and 8 percent of the mean volume entering in this 15-second period.

The no-parking volumes were found to be significantly greater than those with parking at the .05 level for the following time intervals after the beginning of the green; 0-25, 0-30 (entire green period), 10-20, 10-25, 15-25, 15-30, and 20-30 seconds. No significant differences were found in the first 10, 15, or 20 seconds. No other intervals were tested.

Data used in this experiment were collected on four different Sundays. Analysis of variance using data from three of these days showed no significant effect due to day of data taking. Therefore, all data from all days were pooled in reaching the other conclusions of the report.

The traffic lane next to the centerline, designated Lane 1 in this study, was not significantly affected by the parked car. The effect of the parked car was almost entirely on the curb lane, which included traffic moving straight through the intersection and right turning traffic. The latter constituted between $7\frac{1}{2}$ and 10 percent of total traffic under various parking conditions. Significantly more right turns occurred with no parking than with a car parked at 100 feet, and this fact tended possibly to obscure some results.

Analyses were, therefore, conducted using cycles with equal numbers of right turns. The results of these analyses tended to support the conclusions obtained from the pooled data. For cycles with equal numbers of right turns, the effect of parking appeared reasonably well demonstrated. These analyses of parking effect using cycles with equal numbers of right turns were less sensitive than with the pooled data since the degrees of freedom were reduced when cycles were classified by numbers of turns.

These results can be summarized briefly as follows:

1. The effect of a parked car on entering traffic volumes under capacity conditions was found to be significant, but the absolute difference in volumes was small.
2. The effect of parking was not noted in the first half (15 seconds) of the green, but it was pronounced in the last half.
3. The center lane of traffic was not affected by the car parked at the curb.
4. Although approximately one hundred cycles were recorded for each of the two parking positions used, 100 feet and 55 feet, no significant differences in entering traffic volumes were noted between them.

These results appear to support the "Highway Capacity Manual" statement that prohibition of parking for a distance back from the intersection in feet equal to five times the green period in seconds is equivalent to prohibiting parking entirely. There was no parking on the far side of the intersection and the effect of parking here was not studied. A small effect was noted for the 10-20 second interval in this study, and this would not have been the case if the manual's statement were entirely correct. With a car at 100 feet the first 20 seconds of the green should not be affected by parking. No difference was noted here for different parking positions. If the manual's statement were to be borne out fully, the closer parking position would be found more restrictive.

The results of this study indicate that the practical benefits of removing a single car parked in an intersection approach are not great except under certain traffic conditions. The mean difference in entering volumes for the 30-second green period at this intersection was about 1.5 vehicles per cycle with either parking position. This is only 5.5 percent of the entering volume per cycle.

The loss of one and one-half vehicles per cycle from the capacity of the intersection can accumulate an overload in a short time. Ninety vehicles per hour will be unable to enter the intersection, and almost four full cycles will be required to enable these accumulated cars to clear the intersection. The above statement assumes traffic demand will be continuously high during the entire period with at least as many vehicles arriving per minute as are able to enter the intersection without parking.

Removal of a car parked in this position seems justified if and only if every signal cycle is fully occupied. At the southbound approach of Sepulveda to Wilshire, the car has a serious restrictive effect only on Sunday afternoons since normal fluctuations in volumes arriving at the intersection on weekdays will allow any accumulated backlog to clear periodically.

This study was conducted at a single intersection and did not cover the situation in which a single parked car blocks a normally used traffic lane. The results in such a case would be quite different. On Sepulveda Boulevard at Wilshire the single parked car reduced the usable street width but did not reduce the number of lanes normally passing the point at which the car was parked. The results of this study can properly be extended to all situations in which the latter condition prevails: parking does not reduce the number of normally used lanes.

Extending the conclusions of this study suggests that single cars parked in mid-block locations between signalized intersections may have no restrictive effect on capacity. These studies have, however, examined only two distances of parking from the intersection.

Isolated parked cars may present an important safety hazard. This factor has not been considered in the above discussion of the desirability of removing them from the street. In the present study a red flag was attached to the parked car to assure adequate attention and thus help to avoid collisions.

No significant effect was demonstrated as a result of using two different parking positions. Additional experimentation to determine quantitatively the effect of parking positi on volumes entering is needed. Additional data on 100 feet and 55 feet would eventually show a significant difference in restrictive effect if the hypothesis of the "Highway Capacity Manual" is correct. Perhaps more important is to determine the closest distance at which parking can be permitted without its having a significant effect on entering volumes. At the intersection studied this critical distance is greater than 100 feet but has not been established by this experiment. This critical distance is certainly less than 10 car lengths and would probably occur at a somewhat shorter distance. This experiment suggests that large quantities of data will be necessary to draw any definite conclusions about the effect of parking position on entering volume.

The method presented here may prove useful in further studies of intersection operation. The results of this study indicate that recording of entrance times for vehicles represents a suitable method for study of effects of various factors on intersection capacity. The method of field collection of data described here is very simple and reasonably accurate. Additional use of entrance times as basic data will permit important simplification of transcription and analysis of the data.

The practical results of this experiment indicate that parking has a significant but small effect on intersection capacity under conditions such as were studied here. A single car should be removed, if possible, if the approach is overloaded continuously for long periods. Parking a single car certainly should not be permitted where such parking blocks a normally used lane. In this case under study, the intersection was overloaded for less than three hours per week, and parking did not block a normally used lane. Strict prohibition of parking here would be valuable from a capacity standpoint only for a few hours each Sunday afternoon.

In practice, the results of this experiment suggest that traffic engineers and police officials should study traffic conditions very carefully before enforcing parking prohibitions of the tow-away type. The situations in which the single parked car has an important effect are definitely limited.

While only a single street width was studied in this experiment, the findings suggest that interesting results may be found from studies of entering volumes on streets of different widths. The Committee on Highway Capacity presented the relationship of capacity to street width as a smooth curve. The addition of only a few feet of width to the approach of Sepulveda Boulevard studied here, however, might permit use of a full new lane with a resultant large increase in entering traffic volumes. The true relationship of capacity to width may be a type of step function with the sharp capacity increases noted for certain increases of width which permit full utilization of additional lanes. No attempt has been made in this study to evaluate quantitatively the effect of width on capacity. The results suggest, however, that study of this relationship would be desirable.

The effect of parking may also be different on streets of different widths. Certainly this will be the case where the parked car reduced the number of usable lanes. Even where the number of lanes is not reduced, however, street width may be an important factor. Studies similar to this but using wider or narrower streets would be valuable in contributing to the understanding of the effect parking on intersection capacity.

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