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7 Reports

Subject Areas

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Foreword

While our shelves hold much information regarding highway capacity, research in this vital area continues to claim major attention. The several papers in this RECORD are devoted to reporting investigations of matters affecting the capacity of roads, road networks, and intersections. Practicing highway engineers and planners will find these additions to the literature helpful in their consideration of capacity problem areas.

Reilly and Seifert undertook a comparative appraisal of 3 different methods of estimating the capacity of 10 signalized intersections. They found rather large variations in these 3 methods and further found they were not consistent. All 3 methods also suffer from errors of judgment in the difficult choices of capacity factors. Further work is planned.

Tidwell and Humphreys tell of their development of delay charts that can be used to make delay checks for both level of service and failure rate computations. These charts came out of their investigation into the feasibility of utilizing average individual delay as an index of the level of service offered by a signalized intersection.

A systematic analytical procedure to evaluate the effectiveness of freeway improvement plans is described by Makigami and Woodie. Total travel times for both queuing and nonqueuing situations are calculated under given physical and traffic conditions using 3 submodels. Computer operation procedures are also given, and example outputs are compared with data from an existing freeway.

Cooper and Walinchus report their work relative to the Urban Traffic Control Systems Laboratory in Washington, D. C. Their primary concern was to define system objectives and measures of effectiveness as the first step in developing a surveillance methodology. They present a list of candidate measures of effectiveness obtained in their literature search, and recommend only three for the laboratory project.

Field data were collected at 4 unsignalized intersections and analyzed by Surti in an effort to gain an understanding of the operational efficiency of such intersections. A reasonably good correlation between theoretical and observed values of side street delays, queue lengths, and the like was obtained.

The effects on capacity of physical roadway features and of variable traffic factors can to some degree be controlled or regulated, but environmental factors such as weather cannot. Jones, Goolsby, and Brewer correlated rainfall information with traffic data records for a 10-month period on a freeway and concluded that its capacity was reduced to between 81 and 86 percent of dry weather capacity.

Smith, Faulconer, and Smith applied a systems approach utilizing stochastic analysis to evaluate an existing highway network and to evaluate proposed alternative improvements. They also evaluated a technique for analyzing existing traffic conditions through time-lapse photography. They conclude that both the systems approach and the study technique offer some very desirable advantages.

Contents

Capacity of Signalized Intersections

EUGENE F. REILLY and JOSEPH SEIFERT,
New Jersey Department of Transportation

Three methods of estimating capacity at signalized intersections (Highway Capacity Manual, W. Bellis, and R. Dier) are analyzed and compared to a field estimate of capacity (ALE). The Highway Capacity Manual estimate of service volume at the actual load factor (for the field condition) is also compared to the actual peak-hour volume. Using 38 sample approaches, the errors in estimation have been outlined for each of the 3 methods. Overall, the Highway Capacity Manual and Dier methods have errors in excess of ±20 percent for approximately half the sampled approaches. The Bellis procedure (developed in New Jersey, where this study was made) results in errors exceeding ±20 percent for less than 15 percent of the sampled approaches.

THE HIGHWAY CAPACITY MANUAL (HCM) estimates of volume and capacity at signalized intersections are based on several factors. There are at least 2 other, less complex, methods of capacity estimation. One was devised by W. R. Bellis, Director of Research and Evaluation, New Jersey Department of Transportation (2). The other was developed by Robert Dier, Traffic Engineer for Long Beach, California (3).

The purpose of this report is to estimate the capacity of approaches using the HCM method, a modified Bellis method, and the Dier method and to compare these estimates to an empirical (ALE) method. The actual peak-hour volume is also compared to the HCM estimate of peak-hour volume.

The scope of this report includes an explanation of the HCM, Bellis, Dier, and ALE methods, an analysis of the HCM factors, and a detailed examination of the estimated capacities using the various methods.

EXPLANATION OF VARIOUS METHODS

HCM

Approach volume per hour of green, and physical, environmental, and traffic factors are used to determine service volume. A final estimate of service volume, for any level of service, is determined by multiplying the basic approach volume per hour of green by both, the adjustments for these factors, and the G/C (green time/cycle length) ratio for the approach.

Bellis

This method classifies roadways into 4 types. For this report the roads will be defined as follows:

Type I—All central business district (CBD) streets;

Type II—All streets, outside the CBD, that do not fall into the following categories;

Type III—Expressways, arterials, major highways, major streets, and through streets with only right turns at intersections; and

Type IV—Expressways, arterials, major highways, major streets, and through streets with no turns at intersections or with separate phases and turn lanes (including jughandles) provided.

Paper sponsored by Committee on Highway Capacity and presented at the 49th Annual Meeting.

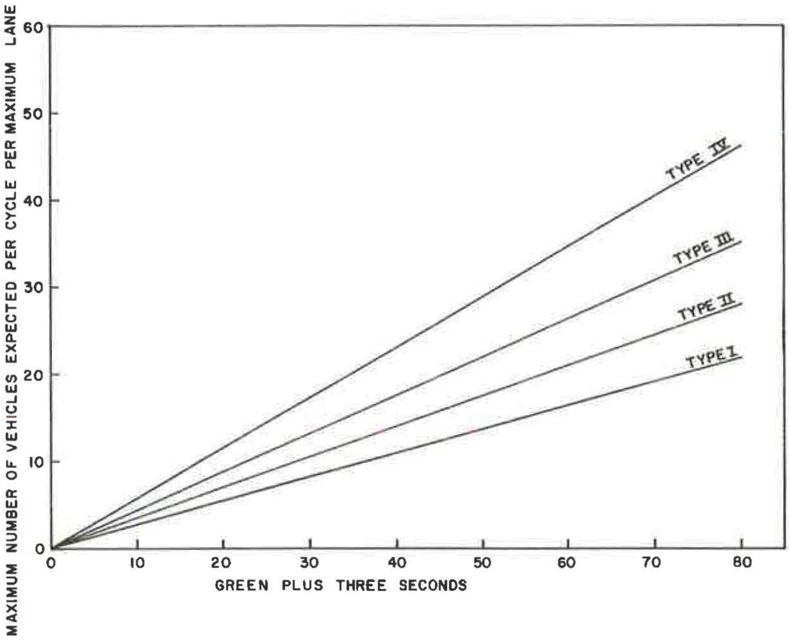
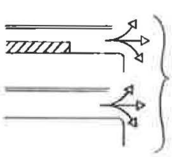
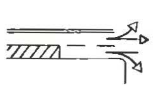


Figure 1. Bellis capacity chart—green plus 3 seconds versus maximum number of vehicles expected per cycle per maximum lane.

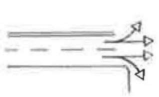
The 4 figures in Bellis' report for road Types I through IV plot 2 variables: the green light required in seconds, and the maximum number of vehicles expected per cycle per maximum lane (Fig. 1). For this study, the green light is interpreted as the green phase plus 3 seconds (for the Bellis procedure only). It is felt that the added amber time gives a more realistic value for the G/C, because a portion of the amber phase (which varies from 3.5 to 6.0 seconds for the approaches used in this study) is used by the drivers. The capacity is estimated by expanding the number of vehicles per cycle to vph and adjusting for lane distribution, turns, and trucks (using the HCM adjustments). Lane distribution is assumed as follows: 2 lanes, 55 and 45 percent; and 3 lanes, 40, 35, and 25 percent. (The maximum lane, which is not necessarily the left or right lane, is given first.) Because the Bellis method was predicated on the through movement of vehicles by lane, the authors used the following criteria to maintain uniformity.



Treated as single lane through the intersection with an adjustment for trucks, but no adjustment for turns.



Treated as a single lane, but adjusted by a factor (1 + proportion of turns).



Treated as 2 lanes, applying the adjustments for turns and trucks.

Dier

The "practical" capacity of over 40 different traffic lane configurations was developed by Robert Dier and expressed in terms of vehicles per second of green. After choosing an appropriate lane configuration from his charts, the rate of flow factor is multiplied by the total green time in the hour. Dier makes provision for grade and truck adjustments. In this study, no grade adjustments were necessary.

ALE

This name is an acronym taken from "average loaded phase expanded" to vehicles per hour. The ALE value is used as an empirical capacity to which the HCM, Bellis, and Dier values are compared. The average number of vehicles for the loaded cycles is used, rather than the maximum or the minimum, because it is felt that this is the most representative value that exists for the loaded phase conditions of trucks, turns, and pedestrian movements. As an example, if there were an average of 20 vehicles per loaded phase and 60 phases per hour, the ALE value would be 20 x 60 or 1,200 vph.

DATA COLLECTION

Data were collected at 38 sites. The results of the first sampling of some sites gave questionable volumes. Hence, 6 of the sites were sampled a second time. In all 6 cases, the capacities yielded by the initial sampling were verified by the second sampling.

Departure data were recorded, by cycle, for each lane. Arrival data were recorded by either minute of time or by cycle. Data were collected for approximately 90 minutes. From these data, the peak hour and the peak 15 minutes within the peak hour were determined. The total number of vehicles, trucks, turns, local buses, and loaded phases were tabulated. Loading was judged using HCM criteria (1, p. 115). To reduce the variability of determining loaded phases, only 2 field parties were used for the study. To train the field crews, the project engineer reviewed loading on a cycle-by-cycle basis under field conditions.

Information on local buses for 10 of the sites was not recorded in the field, but was taken from bus company schedules. For the additional 28 sites, these data were field-recorded.

Loaded cycles with downstream delays were rejected. However, these cycles were used in determining actual peak-hour volume.

Vehicles with over 4 wheels were classified as trucks.

ANALYSIS OF HCM FACTORS

Peak-Hour Factor

Figures 6.5 through 6.10 of the Highway Capacity Manual (1) include tables for the "adjustment for peak-hour factor and metropolitan area size." The adjustments in these tables are the result of the multiple of the 2 factors. If the adjustments are separated into individual factors, it can be seen that the HCM adjustment for peak-hour factor (PHF), not including the metropolitan area size factor, varies with the actual PHF, the type of street such as one-way or parking, and the metropolitan population. Figure 2 shows the variation of this factor with the percentage that the adjustment is greater than the actual PHF.

Figure 2 was derived by taking the HCM adjustment for PHF and metropolitan area size, at a PHF of 1.00, and using this adjustment for metropolitan size only (assuming that an adjustment of 1.00 is used for an actual PHF of 1.00). Each overall adjustment for a particular metropolitan area size is then divided by that found for PHF at 1.00 to give the adjustment for PHF only, without the influence of the metropolitan area size.

The computation of service volume, using the adjustment for the PHF alone, is not appreciably affected unless the PHF is less than 0.89. As the PHF approaches 0.70, the difference between the adjustment and the PHF approaches 20 percent (for 2-way streets, without parking). For example, if the actual PHF is 0.78 (2-way street, parking, population 250,000), the HCM adjustment for the PHF, not including the additional adjustment for metropolitan area size, is 0.813, or 4 percent greater than the actual PHF.

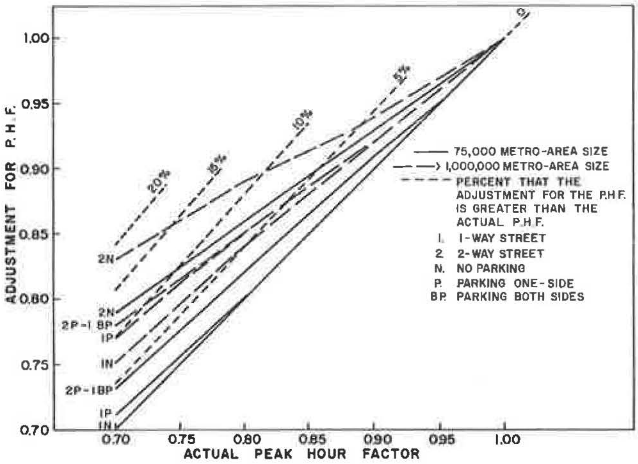


Figure 2. Actual peak-hour factor versus HCM adjustment for PHF.

The load factor at capacity is 1.00; hence, it measures the average number of vehicles departing the intersection during each cycle under the prevailing conditions (provided all cycles are loaded). If every cycle is loaded, a backup of traffic may exist for the entire hour and the "pressure" on drivers to depart the intersection may not be that reflected by the HCM adjustment for PHF.

For load factors less than 1.00, with a PHF less than 1.00, the pressure for drivers to depart the intersection at faster rates may exist to keep the queue size small.

Whatever the reasons for the adjustment factors, Figure 2 shows an increasing PHF adjustment over the actual PHF as (a) the PHF decreases, (b) the metropolitan population increases, and (c) the street goes from one-way operation (from no parking to parking both sides) to two-way operation (from parking to no parking).

Load Factor

The load factor (LF) was determined using the HCM criteria. However, traffic may delay from entering an intersection because of downstream interference during a particular cycle, and the load factor cannot include these cycles. For a precise analysis of load factor and volume, succeeding cycles, which are affected by the previous cycles, should also be eliminated from the data. But the main purpose of collecting data by cycle was to determine the number of vehicles required to load the cycle; hence, the rejected data included only those cycles when downstream delay existed.

Metropolitan Area Population

The HCM estimate of capacity is greatly influenced by the estimator's choice of metropolitan area population. For example, using HCM Figure 6.5, a peak-hour factor of 0.85, and populations of 75,000 and 250,000, the adjustment is either 0.92 or 1.00, a difference of 8.7 percent.

Choosing a realistic population may be easier in western locations where cities are specifically defined. But in northeastern locations, which are part of a megalopolis, the decision is a matter of judgment. The populations used in this study are thus subject to question.

One-Way or Two-Way Streets

A few sites are labeled one-way where the roadways are partitioned by either a median or a center barrier. In this study, where approaches are so divided and there are no left turns at the intersection, the approach is considered to be in the one-way category.

There may be some influence between the 2 opposing directions, especially on roads of minimal median width. The concrete center barrier may also have an adverse effect on drivers.

With or Without Parking

The HCM states that when vehicles are parked within 250 ft of the intersection, the approach should be considered as with parking. However, there are exceptions to this rule (1, p. 114). Parking may exist close to the intersection, and traffic can still make full use of the approach. On the other hand, parking may not be tolerated for progressive signal systems. Again, judgment was used on some of the approaches in this study.

Approach Width

The basic approach volume was extrapolated for 5 of the study approaches, where the width of approach is less than the lowest value shown on the appropriate chart.

There is a shoulder at 3 sites, but no provision is made in the analysis for this extra width. It seems likely that the shoulder may have some effect on capacity.

Green Time/Cycle Length

The green phase alone is used for the green time/cycle length (G/C) computations of HCM service volume and capacity.

Turns and Trucks

Because ALE capacity is determined on the basis of the average number of vehicles serviced per loaded cycle, only these cycles were used to determine the percentage of turns and trucks and the corresponding adjustment factors.

The differences between the peak-hour percentages and the loaded phase percentages of turns and trucks are small, and either one could have been used with minor error.

Local Buses

The exact cycle during which local buses stopped was not field-recorded for 10 of the 38 sites.

Some error may have thus resulted from using the same bus correction for both the peak-hour data and the loaded cycle data. However, the bus correction factor is equal or very close to 1.00 for these 10 sites.

All local bus data were field-recorded for the remaining sites.

The HCM adjustments for "near-side bus stop with parking" give inflated results for the 2-lane, high-turning volume approaches. The presence of one bus per hour on a 2-lane approach with greater than 25 percent turns has the effect of increasing the service volume by 35 percent (bus adjustment factor is 1.35). If the bus stop were removed, the adjustment for local buses would be 1.00.

COMPARISON OF HCM CAPACITY AND VOLUME ESTIMATES WITH ALE AND ACTUAL VOLUME

Volume comparisons are not made for conditions when the LF = 0, because the HCM estimate of volume at LF = 0 is for a condition when one cycle is near loaded. The actual field volume for this condition could be near zero.

For the site characteristics and volumes referred to in the following text, reference should be made to Tables 1 and 2 and the appropriate sketches of the intersections in the Appendix.

One-Way Streets

Volume Comparisons at the Actual Load Factor—Of the 13 samples in this category, taken at 9 different sites, the HCM estimate of volume is either equal to (within ±1 percent) or less than the actual volume for 7 of the 9 locations.

TABLE 1

SAMPLE CHARACTERISTICS

Sample	PHF	Load Factor	Population (000's)	Metro. Location	Bellis Type	Cycle Length	G/C	Width (ft)	1-Way 2-Way	Parking	Loaded Cycle Data: Bus Location	Bus No.	No.	Percent Right Turns	Percent Left Turns	Percent Trucks
1a	0.91	0.90	250	Res.	IV	120	0.618	25	1	None	—	—	27	—	—	11
1b	0.96	1.00	250	Res.	IV	120	0.618	25	1	None	—	—	30	—	—	9
2a	0.79	0.43	75	Res.	IV	90	0.396	25	1	None	—	—	17	—	—	3
2b	0.90	0.90	75	Res.	IV	120	0.364	25	1	None	—	—	27	—	—	7
3	0.92	0.85	250	Res.	IV	90	0.570	30	1	None	Near	10	34	0	—	5
4	0.85	0.00	500	CBD	I	90	0.318	23	1	None	—	—	0	—	—	—
5	0.84	0.27	250	CBD	I	70	0.339	34	1	1 side	Far	15	14	—	—	8
6	0.92	0.28	250	Res.	IV	90	0.571	29	1	1 side	—	—	11	—	0	4
7	0.90	0.76	250	Fringe	II	70	0.322	22	1	1 side	—	—	39	86	—	0
8a[a]	0.92	0.27	250	CBD	I	70	0.460	37	1	1 side	Far	20	14	22	15	2
8b	0.89	0.06	250	CBD	I	70	0.460	37	1	1 side	Far	20	3	18	14	4
9	0.85	0.00	500	CBD	I	90	0.550	40	1	2 sides	—	—	0	—	—	—
10	0.82	0.08	250	Fringe	III	70	0.400	40	1	2 sides	—	—	4	14	0	4
11a	0.80	0.04	250	Fringe	II	70	0.340	38	1	2 sides	Near	0	2	20	30	3
11b	0.85	0.14	250	Fringe	II	70	0.340	38	1	2 sides	Near	7	7	6	34	4
12	0.87	0.21	250	CBD	I	70	0.560	10	2	None	Near	15	11	2	—	3
13	0.75	0.02	250	Outly	III	70	0.600	21	2	None	Near	—	1	11	—	0
14a	0.80	0.12	100	Res.	III	90	0.344	20	2	None	Near	5	5	0	—	3
14b	0.91	0.40	100	Res.	III	90	0.344	20	2	None	Near	0	16	8	—	4
15	0.86	0.23	250	Fringe	II	70	0.514	13	2	None	—	—	12	10	—	1
16	0.89	0.35	250	Res.	III	90	0.700	9	2	None	—	—	14	7	—	0
17	0.79	0.46	250	Res.	II	70	0.450	18	2	None	—	—	24	11	10	6
18	0.87	0.12	250	Fringe	II	70	0.443	26	2	None	—	—	6	2	8	1
19	0.86	0.19	250	Fringe	II	70	0.390	22	2	None	—	—	10	6	6	1
20	0.89	0.78	250	Res.	II	90	0.611	20	2	None	Near	0	31	7	4	2
21[b]	0.87	0.10	250	Fringe	II	70	0.450	26	2	None	—	—	5	15	17	6
22[b]	0.84	0.90	250	Res.	II	70	0.378	9.5	2	None	—	—	46	3	31	2
23[b]	0.81	0.27	250	CBD	I	70	0.460	10	2	None	Far	11	14	—	16	3
24	0.90	0.00	500	Fringe	III	90	0.611	19	2	Yes	—	—	0	—	—	—
25	0.91	0.00	500	Fringe	II	90	0.604	19	2	Yes	—	—	0	—	—	—
26	0.91	0.10	250	CBD	I	120	0.390	25	2	Yes	Far	4	3	12	—	5
27	0.85	0.52	250	Fringe	II	70	0.380	17	2	Yes	Far	6	27	37	0	6
28	0.82	0.63	250	Fringe	II	70	0.490	20	2	Yes	—	—	32	70	15	1
29	0.83	0.41	250	Fringe	II	70	0.380	17	2	Yes	Far	5	21	12	4	3
30a	0.81	0.41	250	Fringe	II	70	0.450	20	2	Yes	—	—	21	17	13	4
30b	0.83	0.58	250	Fringe	II	70	0.450	20	2	Yes	—	—	30	14	16	3
31	0.82	0.41	250	Fringe	II	70	0.510	20	2	Yes	Far	0	21	10	4	4
32	0.87	0.18	250	Fringe	II	70	0.443	25	2	Yes	—	—	9	15	26	1
33	0.86	0.58	250	CBD	I	70	0.460	20	2	Yes	Near	18	30	18	0	5
34	0.86	0.75	500	Fringe	II	90	0.324	21	2	Yes	Near	0	30	25	8	1
35	0.88	0.00	500	Fringe	II	90	0.550	28	2	Yes	Near	0	0	—	—	—
36	0.81	0.18	250	Fringe	II	70	0.510	20	2	Yes	Near	6	9	7	7	1
37	0.94	0.35	250	Fringe	II	70	0.600	22	2	Yes	Near	3	18	2	7	1
38	0.85	0.88	250	Fringe	II	70	0.400	22	2	Yes	Near	6	45	9	3	5

[a]Policeman enforcing controls in intersection. [b]Exclusive of left-turn lane.

In one of the 2 cases (site 3) where the HCM estimate is greater than the actual volume, it should be noted that the basic HCM approach volume is for a 30-ft approach (compared to 25 ft for sites 1 and 2). It would appear that the additional 5-ft width of approach, for a roadway marked for 2 lanes, should only increase the basic approach volume (over a 25-ft width) by about 200 vph of green rather than the 400 vph of green indicated by Figure 6.5 of the Highway Capacity Manual (1).

The other location, site 11 (samples 11a and 11b) where the HCM estimate was higher than the actual volume, has a near-side bus stop, with parking. Because few local buses stop, and there are greater than 25 percent turning movements, the HCM adjustment for this factor is approximately 1.25. However, if the bus stop were removed, the HCM adjustment would be reduced to 1.00, yielding HCM estimates of volume below those found in the field. This is the first indication that the HCM adjustments for near-side bus stops, with parking, could be extremely high. Sites 33 through 38 (to be discussed later in this report) give similar results.

Sample 8a shows a +3 percent difference, which may be explained by the influence of the policeman within the intersection during the study. His presence may have slowed the traffic as it came through the intersection.

Capacity Comparison—The HCM estimate of capacity ranges from -33 to +40 percent of the ALE capacity.

Site 11, which is composed of samples 11a and 11b, has the largest differences, +40 and +33 percent. This site has a near-side bus stop, but few local buses stop. The HCM adjustment for this case is 1.24. If the bus stop were removed, the HCM

TABLE 2

SAMPLE VOLUMES AND CAPACITIES

Sample	PH Vol.	HCM Vol.	Percent Diff.[a]	Capacity						
				ALE	HCM	Percent Diff.[b]	Bellis	Percent Diff.[b]	Dier	Percent Diff.[b]
1a	1,934	1,780	-8	1,950	1,820	-7	2,170	11	2,190	12
1b	1,970	1,980	1	1,970	1,980	1	2,180	11	2,200	12
2a	1,448	910	-37	1,520	1,020	-33	1,560	3	1,430	-6
2b	1,330	1,000	-25	1,340	1,020	-24	1,370	2	1,300	-3
3	1,864	2,060	11	1,930	2,100	9	2,160	12	2,070	7
4	473	550	NA	—	—	—	—	—	—	—
5	700	640	-9	880	790	-10	600	-32	1,220	39
6	1,450	1,310	-9	1,640	1,600	-2	2,160	32	2,020	23
7	413	390	-5	460	410	-11	460	0	530	15
8a[c]	1,050	1,080	3	1,190	1,380	16	1,050	-12	1,650	39
8b	1,090	940	-14	1,250	1,310	5	1,030	-18	1,650	32
9	571	980	NA	—	—	—	—	—	—	—
10	760	710	-7	1,010	1,100	9	1,220	21	1,330	32
11a	590	670	14	770	1,080	40	770	0	1,120	45
11b	620	760	23	820	1,090	33	780	-5	1,120	37
12	470	480	2	620	680	10	560	-10	1,000	61
13	690	990	43	1,390	1,480	6	1,780	28	2,140	54
14a	867	590	-32	1,140	780	-32	1,060	-7	1,220	7
14b	900	690	-23	1,110	810	-27	1,010	-9	1,220	10
15	460	820	78	600	950	58	660	10	910	52
16	904	760	-16	1,040	970	-7	1,160	12	1,240	19
17	460	690	50	520	820	58	580	12	620	19
18	640	1,060	66	1,010	1,380	37	1,020	1	1,180	17
19	510	730	43	850	1,020	20	940	11	1,000	18
20	1,070	1,360	27	1,120	1,530	37	870	-22	1,630	46
21[d]	720	1,140	58	1,020	1,320	29	1,000	-2	1,510	48
21L[e]	180	100	-44	210	100	-52	—	—	180	-14
22[d]	530	540	2	550	580	5	500	-7	690	25
22L[e]	170	100	-41	260	100	-62	—	—	150	-42
23[d]	360	400	11	450	600	33	450	0	880	95
23L[e]	80	170	113	110	170	55	—	—	30	-73
24	345	1,050	NA	—	—	—	—	—	—	—
25	506	1,180	NA	—	—	—	—	—	—	—
26	468	550	18	560	670	20	390	-30	340	-39
27	380	440	16	450	470	4	480	7	620	38
28	608	490	-19	670	550	-18	660	1	650	-3
29	355	470	32	450	490	9	500	11	500	11
30a	460	470	2	570	550	-4	590	4	590	4
30b	470	500	6	560	550	-2	600	7	590	5
31	570	710	25	700	860	23	730	4	670	-4
32	670	620	-7	900	740	-19	860	-4	890	-1
33	530	690	30	570	790	39	480	-16	410	-28
34	404	630	56	430	690	60	440	2	430	0
35	640	1,270	NA	—	—	—	—	—	—	—
36	563	800	42	680	970	43	750	10	680	0
37	880	1,040	18	950	1,280	35	890	-6	800	-16
38	540	720	33	550	750	36	600	9	530	-4

NA = not applicable.

[a]Based on PH volume.
[b]Based on ALE.
[c]Policeman enforcing controls in intersection.
[d]Exclusive of left-turn lane.
[e]Separate left-turn lane.

adjustment would be reduced to 1.00, with a resulting difference of +12 and +9 percent with ALE.

Sample 8a has the next largest positive difference, +16 percent. However, this site had a policeman enforcing the signal controls during the period of study. Thus, his presence may have impeded the flow of vehicles to some degree.

The capacity comparison for sample 3 is similar to the volume comparison of the previous section. It would again appear that a 400 vph of green increase in basic approach volume (as indicated by the HCM) of a 30-ft wide roadway over a 25-ft wide roadway (both marked for 2 lanes) is higher than the capacity attained in the field.

Of the 6 samples where the HCM estimates of capacity are higher than ALE, 5 of the samples have parking on either one or both sides of the one-way approach.

Two-Way Streets With No Parking

Volume Comparisons at the Actual Load Factor—Thirteen samples were studied in this category using 12 different approaches. Of the 12 approaches, the HCM estimate of volume was high at 10 of them (ranging from +2 to +78 percent).

For those approaches marked for 2 lanes, the following tabulation shows how the percentage difference in HCM estimate increases with an increase in width of approach. (All samples have a metropolitan location factor of 1.25.)

Sample	Width	Multiple of Turn Adj.	Percent Diff.
14a	20	1.16	-32
14b	20	1.11	-23
20	20	1.07	+27
13	21	1.07	+43
19	22	1.00	+43
21	26	1.08	+58
18	26	1.07	+66

A similar trend is noted for the one-lane approaches. (All samples have a metropolitan location factor of 1.25, except samples 23 and 12, which have a factor of 1.00.)

Sample	Width	Multiple of Turn Adj.	Percent Diff.
16	9	1.38	-16
22	9.5	1.48	2
23	10	1.56	11
12	10	1.51	2
15	13	1.31	78
17	18	0.97	50

The only factor of importance that distinguishes samples 14a, 14b, and 16 (the sample approaches at which the HCM estimate was low) from the others is that site 14 is on the approach to a bridge entering a city, and site 16 is the departure of a bridge leaving a city. It is difficult to determine from these listings the exact cause for the HCM differences. The 2 factors, width (hence, basic approach volume) and turning movements (the break-off width in the HCM, for significant differences in turn factors, is 16 ft), are present simultaneously. For the volume comparisons, there is also a difference in load factor between the separate samples. This latter factor is removed in the capacity comparisons of the next section.

There are 3 "separate left turn lane" samples in this category. The HCM estimate of the volumes for the samples shows little similarity with the actual left-turn volumes.

Capacity Comparison—The results of the HCM estimate of capacity (LF = 1.00) are similar to those of volume. The HCM estimate of capacity is again high for 10 of the 12 sites studied, and again exhibits a tendency for this difference to get proportionately larger as the width of roadway increases (while keeping the number of marked lanes constant).

For the approaches marked for 2 lanes, all samples have a metropolitan location factor of 1.25.

Sample	Width	Multiple of Turn Adj.	Percent Diff.
14a	20	1.15	-32
14b	20	1.11	-27
20	20	1.08	+37
13	21	1.10	+6
19	22	1.06	+20
21	26	1.08	+29
18	26	1.06	+37

For the one-lane approaches, all samples have a metropolitan location factor of 1.25, except samples 23 and 12, which have a factor of 1.00.

Sample	Width	Multiple of Turn Adj.	Percent Diff.
16	9	1.38	-7
22	9.5	1.48	+5
23	10	1.56	+33
12	10	1.51	10
15	13	1.30	+58
17	18	1.00	+58

Again, the precise nature of a revised shape of the LF = 1.00 curve in the HCM cannot be determined because of the simultaneous influence of the turning movements. But even with a controlled study, the results may only be applicable to New Jersey.

Two-Way Streets With Parking

Volume Comparisons at the Actual Load Factor—The HCM estimate of volume is higher than the actual volume for 11 of the 13 samples in this category. Five of these 11 samples have a near-side bus stop. If the adjustment for the bus stop is reduced to 1.00 from a range of 1.08 to 1.35, the HCM estimate would still be higher than the actual volume by a range from +9 to 25 percent.

For the 2 samples, 28 and 32, where the HCM estimate of volume is less than the actual volume, the turning traffic is 83 and 33 percent respectively of the approach

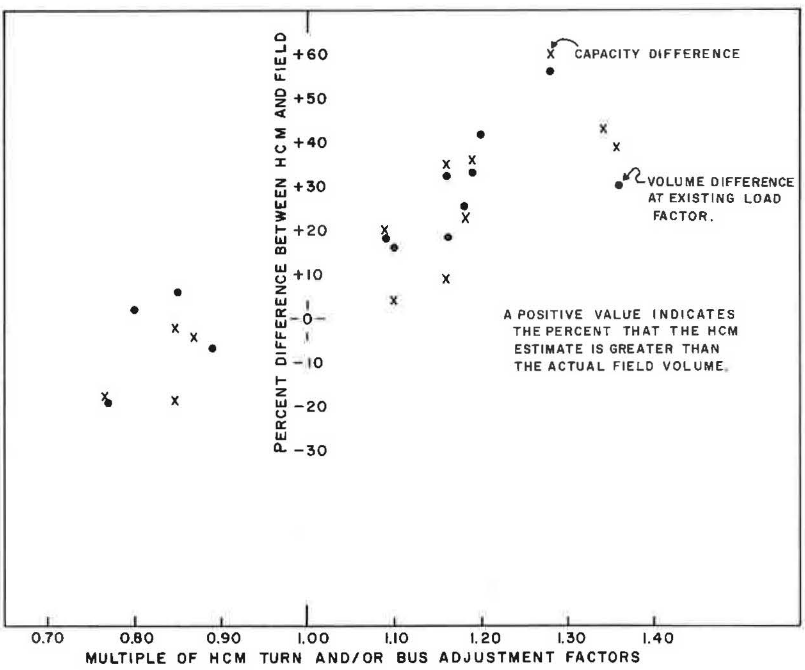


Figure 3. Multiple of HCM turn and/or bus adjustment factors versus percentage difference between HCM and field—2-way streets with parking.

volume. For those samples where there is no near-side bus stop (26 through 32), the positive and negative differences between the HCM estimate and the peak-hour volume closely approximate the multiple of HCM turn factors. When the bus stop factor is included with the multiple of the turn factors (for samples 33, 34, 36, 37, and 38), a similar trend is evident.

Capacity Comparison—As with the volume comparison, the positive or negative difference in the HCM estimate of capacity with ALE is closely related to the multiple of turn and/or bus adjustment factors. Figure 3 shows the variation of the multiple of these adjustment factors with the percentage difference between the HCM estimate and the actual peak-hour volume and ALE.

COMPARISON OF BELLIS CAPACITY ESTIMATES WITH ALE

The Bellis procedure does not estimate volumes between the lowest load factor (one loaded cycle) and capacity (all cycles loaded). Because there is just one sample in the study that had one loaded cycle, only a comparison of ALE and the Bellis estimate of capacity will be made.

For the 7 Type I samples, 6 have values between 10 and 32 percent below the ALE capacity. In one case the Bellis capacity and the ALE capacity are equal. These results indicate that the Bellis method underestimates capacity for CBD locations.

For the Type II samples, the Bellis capacities range from below to above the ALE values. Six samples are below ALE by 2 through 22 percent. For 4 samples, the Bellis estimate equals the ALE value (±1 percent). For another 11 samples, Bellis overestimates capacity by 4 through 12 percent. These samples include both parking and no parking conditions, which are not differentiated by the Bellis method.

For the Type III samples, 2 underestimate capacity by 7 and 9 percent, and 3 overestimate capacity by 12 through 21 percent.

For the Type IV samples, all 6 estimates exceed capacity by 2 through 32 percent.

Perhaps these results suggest slight revisions to Figure 1. The slope of the Type I line could be raised (because Type I capacities are underestimated), and the slopes of the Types III and IV lines could be lowered. The mean differences between the Bellis estimate and ALE are as follows:

Bellis Type	No. Samples	Mean Percent Diff.	Standard Deviation
I	7	-17	±10.4
II	21	2	±7.9
III	5	9	±14.8
IV	6	11	±9.9

COMPARISON OF DIER CAPACITY ESTIMATES WITH ALE

Dier capacities were divided into groups based on various lane configurations (Table 3).

Optional Right-Turn and Through Lane

Dier overestimates capacity for 5 of the 7 samples. The 2 samples that are underestimated are located in the CBD.

Left-Turn Only Lane

Dier underestimates capacity for the 3 samples. The California streets used to determine the Dier flow rates are wider than the streets used in this study. (Left turns across a wide street may be more difficult to make and, hence, have a lower flow rate than on a narrower street.)

TABLE 3

RATE OF FLOW AND PERCENTAGE DIFFERENCE FOR DIER METHOD

Lane Configuration	Sample	Vehicles per Second[a]	Percent Diff.[b]
Optional right-turn and through lane	15	0.49	52
	16	0.49	19
	22	0.49	25
	7	0.46	15
	27	0.46	38
	26[c]	0.25	-39
	33[c]	0.25	-28
Left-turn only lane	21L	0.11	-14
	22L	0.11	-42
	23L[c]	0.02	-73
Optional left-turn, right-turn, and through lane	17	0.39	19
	28	0.37	-3
	29	0.37	11
	30a	0.37	4
	30b	0.37	5
	31	0.37	-4
	34	0.37	0
	36	0.37	0
	37	0.37	-16
	38	0.37	-4
Through lane	12[c]	0.50	61
	23[c]	0.50	95
	1a	1.02	12
	1b	1.02	12
	2a	1.02	-6
	2b	1.02	-3
	3	1.02	7
	5[c]	0.96	39
	6	0.99	23
Optional right-turn and through lane, plus through lane	10	0.93	32
	13	0.99	54
	14a	0.99	7
	14b	0.99	10
	21	1.00	48
Optional right-turn and through lane, plus optional left-turn and through lane	11a	0.92	45
	11b	0.92	37
	18	0.74	17
	19	0.71	18
	20	0.74	46
Optional left-turn and through lane, plus right-turn only lane	32	0.56	-1
Optional left-turn and through lane, through lane, plus optional right-turn and through lane	8a[c]	1.00	39
	8b[c]	1.00	32

[a]Flow rate for individual lane configurations taken from Dier (3). [b]Difference between Dier and ALE capacities. [c]CBD location.

Optional Left-Turn, Right-Turn, and Through Lane

Dier's estimate of capacity is within ±20 percent of ALE for the 10 samples in this category. The average error is 1.2 percent with a standard deviation of ±8.4 percent.

Through Lane

For the 4 sites outside the CBD, Dier overestimates the capacity of three of them and underestimates one.

The site, which is underestimated, is a high-type roadway. (However, the estimate of capacity at another high-type location, site 1, is high by 12 percent.)

For the 3 CBD samples, Dier overestimates capacity by at least 39 percent. One of these samples has an exclusive bus lane, and another has an exclusive left-turn lane.

Optional Right-Turn and Through Lane, Plus Through Lane

Dier overestimates capacity for all 5 samples. This seems reasonable, because the optional right turn and through lane estimates, discussed earlier, for non-CBD areas are greater than ALE. The capacity of the through lanes is also overestimated by Dier.

Optional Right-Turn and Through Lane, Plus Optional Left-Turn and Through Lane

Dier overestimates capacity for all 5 samples. Consistent with the optional right turn and through lanes, discussed earlier, the right turn and through lanes of this category were also overestimated by Dier. Hence, his estimation of the capacity of these approaches is high.

Optional Left-Turn and Through Lane, Plus Right-Turn Only Lane

In this single case, the Dier estimate of capacity is equal to ALE.

Optional Left-Turn and Through Lane, Through Lane, Plus Optional Right-Turn and Through Lane

Both samples overestimate capacity for this CBD location. When individual lanes are examined, Dier's estimate of capacity for the through lane exceeds ALE by approximately 70 percent. For through lanes, discussed earlier, the Dier estimate of capacity exceeded ALE by 39, 61, and 95 percent (for CBD locations).

CONCLUSIONS

Most of the sampled data for this study were collected in or near Trenton, New Jersey. An overall comparison of the accuracy of capacity estimation for the 3 methods studied in this report is evident from the data shown in Figure 4.

Regional differences in driver characteristics may be inferred from the positions of the 3 curves. As may be expected, the Bellis method of capacity estimation is the most accurate, probably because this method was developed in the state of New Jersey.

Because of the uniqueness of each of the methods, an individual analysis is made of their effectiveness.

HCM

For approximately half the study samples, the HCM method yields estimates in excess of ±20 percent of the peak-hour volume and ALE values.

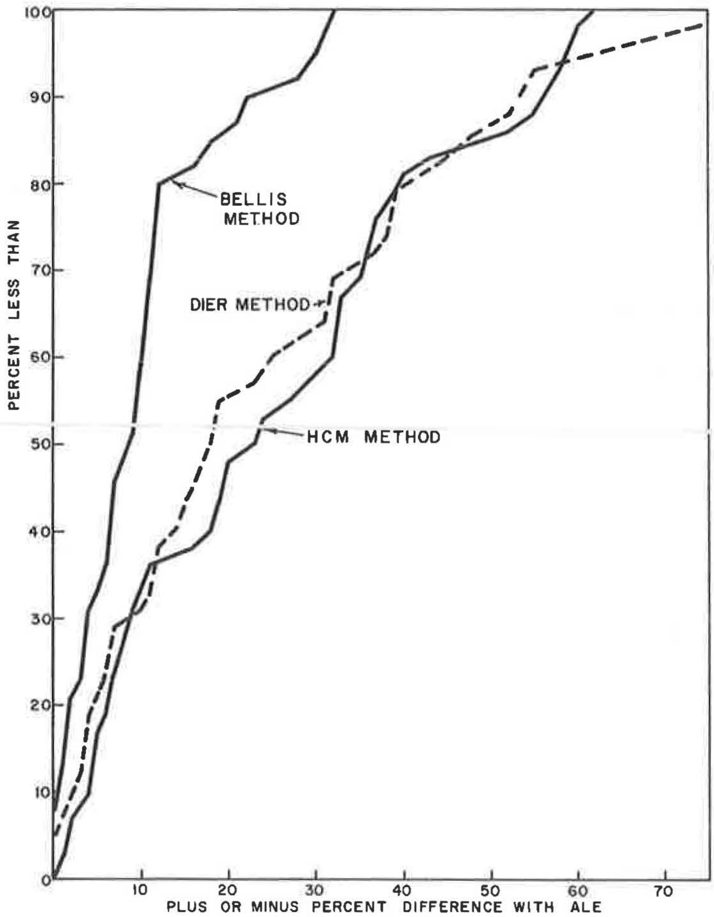


Figure 4. Cumulative frequency curves of percentage differences of methods of capacity estimation with ALE.

With the limitations of 38 sites (42 samples), the main reasons for the inaccuracy of this method appear to be the following:

1. The adjustment factor for near-side bus stops on 2- and 3-lane streets, with parking, gives inflated values for volume and capacity estimates.
2. The basic approach volume is based on width of approach, rather than number of lanes. The fact that this procedure may lead to erroneous results is evident on 2-way streets, without parking.
3. To some extent the turn adjustment factors for narrow approaches (between 10 and 15 ft) may be too extreme.
4. The computation of volume for the exclusive left-turn lane, while rational, is far from accurate for the 3 samples studied in this report.

Bellis

Of the 4 types of streets that are estimated under the Bellis procedure, the most variation is found in Type III (the standard deviation of the estimate for the sampled data is ±14.8 percent).

For consistency in the estimate, the Type I streets are estimated low, and the Type IV streets are estimated high.

The estimate of capacity for Type II streets is the most accurate, with over 95 percent of the samples (20 of 21) within ±12 percent of ALE.

Dier

As with the Bellis method, the Dier procedure uses a rate of flow by lane.

The errors of the estimate of capacity by this method can be tentatively reduced to 3 primary lane configurations.

1. The estimate of capacity for the optional right turn and through lane (outside the CBD) is consistently high.
2. The left-turn only lane estimate has been developed by Dier using wide streets and, hence, is low for the samples of this study.
3. The estimate of capacity for the through lane in the CBD locations is consistently high.

ACKNOWLEDGMENT

This study was conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads. The opinions, findings, and conclusions expressed in this paper are those of the authors and not necessarily those of the Bureau of Public Roads.

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2. Bellis, W. R. Capacity of Traffic Signals and Traffic Signal Timing. HRB Bull. 271, 1960, pp. 45-67.
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Appendix

The various kinds of intersections are shown in Figures 5 through 9.

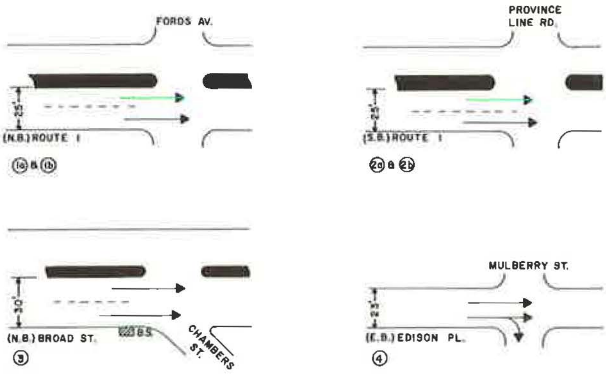

Figure 5. One-way, no parking.

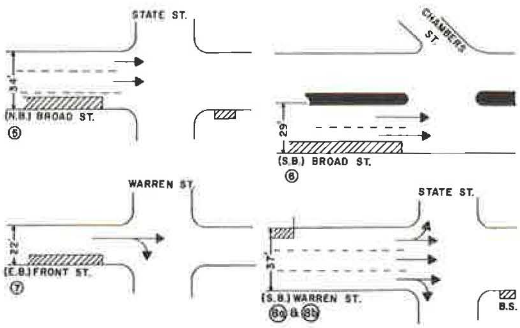

Figure 6. One-way, one side parking.

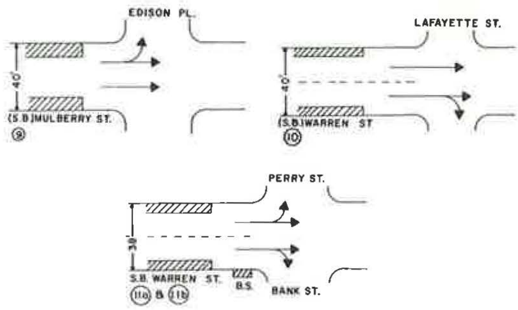

Figure 7. One-way, two side parking.

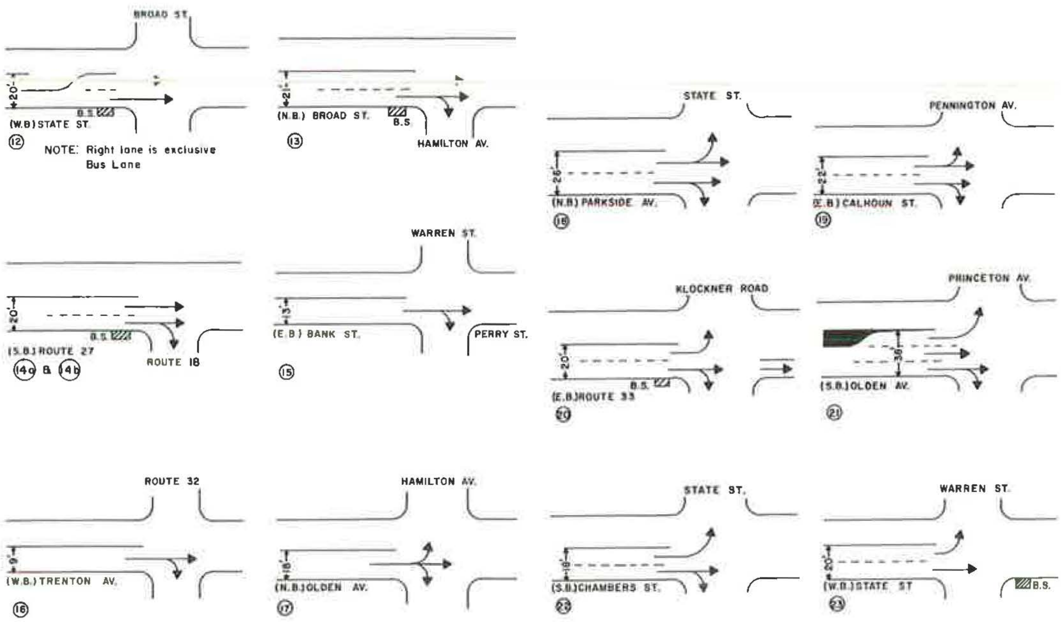

Figure 8. Two-way, no parking.

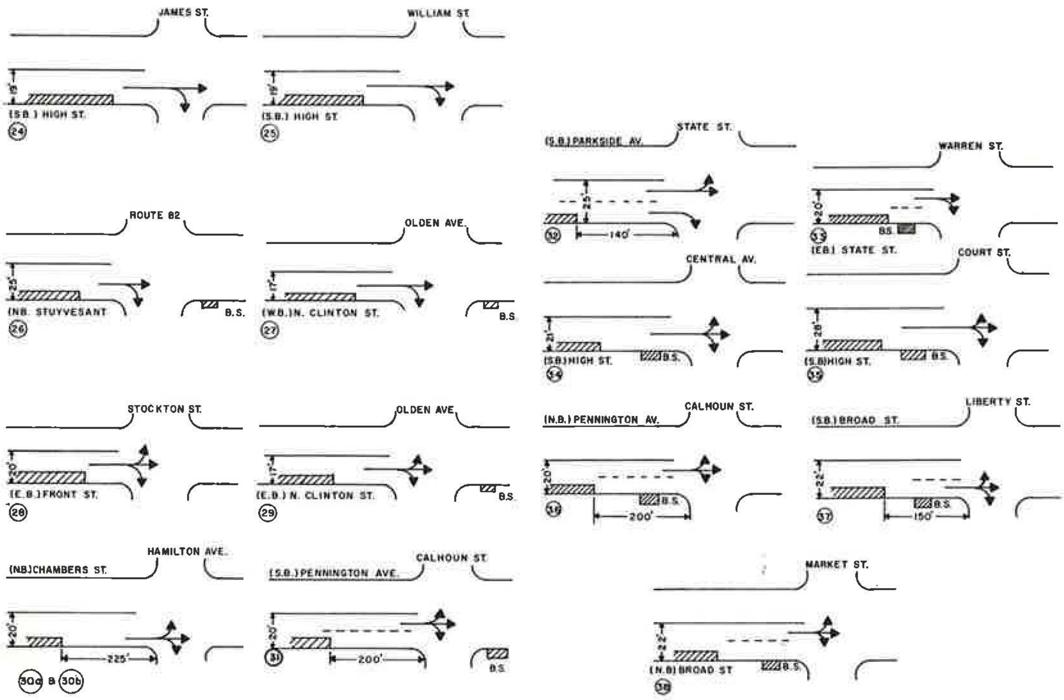

Figure 9. Two-way, parking.

Relation of Signalized Intersection Level of Service to Failure Rate and Average Individual Delay

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The most widely used signalized intersection design procedure is that found in the 1965 Highway Capacity Manual. This manual utilizes vehicles per hour of green as an indicator of the level of service offered by an installation. An alternative intersection solution procedure utilizes the percentage of cycle failures as an indicator of fixed time intersection performance. The research reported here indicates that failure rate does not apparently correlate with level of service as defined by the Highway Capacity Manual. The 2 indexes have a varying relationship. Charts and tables are presented for use in conversion. For a constant service level, a low approach volume will allow higher failure rates than may be tolerated for high volumes.

In line with prior work in this field, an investigation was made into the feasibility of utilizing average individual delay as an index of the level of service offered by a signalized intersection. Delay was related to failure rate and charts prepared for various cycle lengths utilizing Webster's equation for average individual delay. These charts indicated that failure rate and delay also have a varying relationship. This relationship was oriented in a different manner from the preceding relationship. Using constant delay lines, higher failure rates were allowed for high volumes. A combined plot of level of service lines and delay lines indicates an apparent divergence of level of service and average individual delay. This divergence demonstrates that, although an intersection may satisfy the Highway Capacity Manual's criteria of a given vehicles per hour of green for a desired level of service, average individual delay may vary considerably depending on arrivals, cycle length, and length of green time.

•THE 1965 HIGHWAY CAPACITY MANUAL (1) introduced the concept "level of service" for both uninterrupted flow conditions and street intersections with signalized control. For uninterrupted flow conditions, speed and the volume/capacity ratio were selected as measures of the level of service offered by a facility. Chapter 6 of the 1965 Highway Capacity Manual discusses the level of service concept as it relates to at-grade intersections or interrupted flow (1, p. 130):

> Inasmuch as level of service is described in terms of driver satisfaction, the substitute measure should be some factor that the driver himself sees and interprets in terms of degree of congestion. Of the several factors that have been discussed in the previous section, probably load factor is the

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most evident to the average driver. Hence it is the best measure of the level of service at individual intersections with no or only average signal coordination.

The 1965 Highway Capacity Manual uses load factor as the determinant for the various service levels. The computational procedure for determining the level of service offered by a signalized intersection is basically the same as that presented by the 1950 Highway Capacity Manual (2), in that capacity at various levels is obtained from charts in terms of a specific number of vehicles per hour of green after applying appropriate correction factors. The 1950 Highway Capacity Manual used this technique for determining basic, possible, and practical capacities of an installation. Possible capacity required a continual backlog of vehicles, hence a high load factor and a low level of service. Practical capacity was defined as the volume where most vehicles would clear the intersection without waiting for more than one complete signal cycle, hence a lower load factor and higher level of service.

FAILURE RATE

Drew and Pinnell (3) have presented evidence that peak-period traffic flow approaching a signalized intersection is accurately defined by the Poisson probability distribution. They have utilized this finding to develop a design procedure using the percentage of cycle failures for fixed time installations. A cycle failure is defined as any cycle during which approach arrivals exceed the capacity for departures. Briefly this procedure assumes that departures during a green phase of a cycle may be computed by the equation

$$X = \frac{G - (K - D)}{D}$$

where

X = maximum departures per lane for one approach during a green phase;
G = length of the green phase of the cycle in seconds including yellow time;
K = sum of starting delay and time for last vehicle to cross intersection; and
D = average minimum headway in seconds.

For design purposes, the K factor was determined to be 6 seconds for the average intersection and D, the average minimum headway, was established as 2 seconds. Using this equation and constants with the cumulative Poisson probability distribution, a design chart was established (Fig. 1). This chart is entered with the average number of arrivals per approach lane per cycle and the green time allocated to that approach lane. Noting the intersection of the 2 variables, a determination may then be made of the probability that more vehicles will arrive at the approach than may pass through the installation during a green phase for that approach. Of course, this neglects the carry-over of queues from one cycle to the next, inasmuch as only new arrivals are being considered. For high traffic volume it overstates the probability that a particular vehicle will clear the intersection during its first cycle at the signal.

In their development of the failure rate design procedure, Drew and Pinnell (3) have derived a multiple regression equation for the determination of the peak-period flow rate when the peak-hour flow rate is known. Their equation is as follows:

$$Y' = 1.225 - 0.000135\, X_1 \pm (0.1X_2 - 0.00003X_3)$$

where

Y' = factor to be applied to approach peak-hour flow rate to determine peak-period flow rate for the approach;
X_1 = population 1,000;
X_2 = ratio distribution = $\frac{\text{distance between intersection and CBD}}{\text{distance from CBD to city limits}}$; and
X_3 = peak hourly volume per approach.

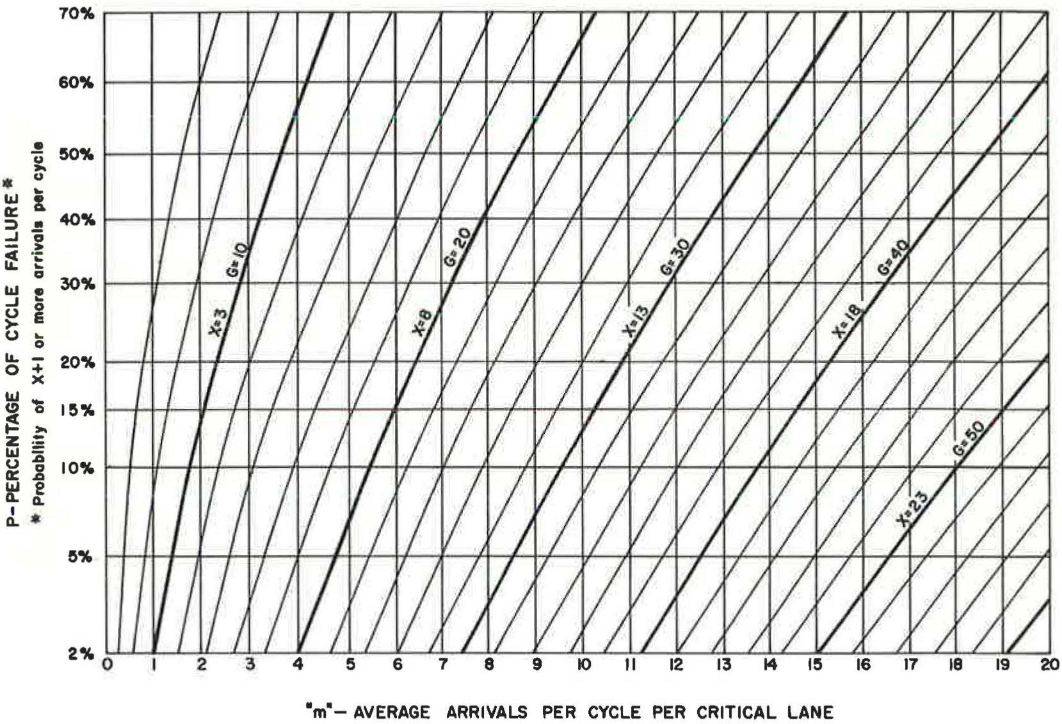

Figure 1. Failure rate chart from Drew and Pinnell (3).

The plus or minus factor in this equation is positive for morning flow and negative for evening flow.

Drew and Pinnell (3) attempted to develop an equation for the duration of the peak period. However, this relationship proved to be statistically unreliable.

Drew and Pinnell (3) have utilized these concepts to develop a design method for high-type signalized intersections where every movement has a separate signal phase. Knowing the peak-hour flow rate, the peak period flow rate may be determined using this multiple regression equation. A trial design of the intersection layout is then made. From the number of lanes allocated to an approach, the average number of vehicles per cycle per approach lane for each phase may be determined. For a 4-leg high-type intersection, there may be 4 separate phases: 2 through and 2 left turn phases. After determining the average lane arrivals per cycle for each phase and choosing for each phase the lane with the highest average arrivals, which is referred to as the critical lane volume for that phase, a cycle length is chosen. Green times are then allocated to each phase such that when the design chart is entered with the green time and average arrivals, a desired failure rate for all phases is obtained. Of course, the summation of the green times for all phases must equal the cycle length. However, some determination must be made as to what percentage failure rate is acceptable for design practice. Drew and Pinnell state (3):

> Although additional research is needed in deciding just what percentage of failures may reasonably be allowed, it seems that a level of 30 to 35 percent during the peak period represents a practical design level (remembering that this would be only about 10 to 15 percent of the peak hour).

The writers have solved intersection problems utilizing both the procedure in the 1965 Highway Capacity Manual and Drew and Pinnell's failure rate procedure. In some instances, a low level of service at the 30 to 35 percent failure rate level has been obtained. Although the 2 concepts are different in approach, in that one is subjective and based on thousands of actual observations and the other is mathematical, an investigation into their relationship appeared worthwhile. The relation presented will be for only high-type intersections such as might be solved by the failure rate method.

RELATION OF LEVEL OF SERVICE TO FAILURE RATE

In order to provide some common ground for relating the 2 procedures, nomograms developed by Leisch (4) were utilized for determining allowable vehicles per hour of green for various levels of service and approach lanes. Leisch's nomograms were developed from the Manual's procedures, but they utilize an MP correction factor that combines the Manual's peak period and population factors. To determine the MP factor, one needs only to know the population of the city where the installation is to be made. Of course, if the peak-period factor as defined by the Manual is known, it and the city population can be used to obtain a factor for use in the nomograms. This relationship is shown in Figures 6.5 through 6.9 in the 1965 Highway Capacity Manual. These nomograms were utilized to develop the allowable vehicles per hour of green for various intersection locations and approach lane arrangement. For comparison with Drew and Pinnell's method, 12-ft through lanes and 10-ft left turn lanes were assumed because this lane width would probably be used at the type of intersections considered by Drew and Pinnell. The allowable vehicles per hour of green for various numbers of approach lanes and locations within a city as obtained from Leisch (4) are given in Table 1. The charts for one-way operation (two-way charts used for single through lane approaches) and no parking were chosen because it was considered that these conditions best approximated approach operation at a high-type intersection where each movement has a separate signal phase. Leisch's turning lane charts were used directly. No truck, bus, or turn factors other than those built into the charts are considered for this general case. The definitions for central business district (CBD), fringe area, outlying

TABLE 1

VEHICLES PER HOUR OF GREEN (VPHG) PER APPROACH LANE AND VEHICLES PER SECOND OF GREEN (VPSG) FOR VARIOUS LOCATIONS WITHIN A CITY FOR DIFFERENT LEVELS OF SERVICE

Area	Level of Service	Single Left Turn (10 ft)[a]	Double Left Turn (10 ft)[a]	1 Through Lane (12 ft)[b]	2 Through Lanes (24 ft)[c]	3 Through Lanes (36 ft)[c]	4 Through Lanes (48 ft)[c]
CBD	A	810 vphg	729	884	1,045	987	987
		0.225 vpsg	0.212	0.245	0.290	0.274	0.274
	B	810	729	936	1,068	1,008	1,008
		0.225	0.212	0.260	0.297	0.280	0.280
	C	900	810	1,040	1,100	1,050	1,050
		0.250	0.225	0.289	0.306	0.292	0.292
	D	1,080	972	1,186	1,200	1,122	1,155
		0.300	0.270	0.329	0.333	0.313	0.321
	E	1,170	1,053	1,248	1,242	1,175	1,208
		0.325	0.293	0.347	0.345	0.326	0.335
OBD or fringe	A	810 vphg	729	1,020	1,140	1,066	1,058
		0.225 vpsg	0.212	0.283	0.317	0.296	0.294
	B	810	729	1,080	1,164	1,088	1,080
		0.225	0.212	0.300	0.324	0.302	0.300
	C	900	810	1,200	1,200	1,132	1,125
		0.250	0.225	0.333	0.333	0.315	0.313
	D	1,080	972	1,368	1,320	1,214	1,239
		0.300	0.270	0.379	0.366	0.338	0.344
	E	1,170	1,053	1,440	1,356	1,270	1,294
		0.325	0.293	0.400	0.377	0.353	0.360
Residential	A	810 vphg	729	1,020	1,236	1,175	1,175
		0.225 vpsg	0.212	0.283	0.343	0.326	0.326
	B	810	729	1,080	1,260	1,200	1,200
		0.225	0.212	0.300	0.350	0.333	0.333
	C	900	810	1,200	1,300	1,250	1,250
		0.250	0.225	0.333	0.361	0.347	0.347
	D	1,080	972	1,368	1,430	1,338	1,375
		0.300	0.270	0.379	0.397	0.372	0.382
	E	1,170	1,053	1,440	1,470	1,400	1,438
		0.325	0.293	0.400	0.408	0.389	0.399

Note: These data are for a city population of 250,000. The factors in Table 2 should be used to adjust the values in this table for other populations.
[a]Data from Leisch's turn lane charts (4). [b]Data from Leisch's two-way charts, no parking (4). [c]Data from Leisch's one-way charts, no parking (4).

business district (OBD), and residential area are found in the Manual and Leisch's paper, but these are basically self-explanatory. This table was developed assuming an MP of 1.00, which corresponds to a city of 250,000 population. Correction factors for use with cities of different populations will be discussed later.

In relating the level of service and failure rate procedures, the first step is to determine the maximum allowable average arrivals to an approach lane of an installation with a given cycle length and a given green time, while holding the vehicles per hour of green constant. This may be done by converting the vehicles per hour of green to vehicles per second of green per lane. The length of green time for the particular phase may be multiplied by the allowable vehicles per second of green time per approach lane to determine the maximum allowable average arrivals per cycle to an approach lane.

It is to be noted that this is in reality the basic operation performed while using the Manual's design procedure. Assume that an approach volume is given and it is desired to know the green requirements for a particular level of service. The ratio of approach volume to the given vehicles per hour of green determines the required signal split. The equations for this operation would be as follows:

$$g/c = \text{vph}/\text{vphg}$$

$$\text{vph} = \text{vphg}(g/c)$$

if m = maximum arrivals per cycle, then

$$m = \frac{\text{vph}}{\text{cycles/hour}} = \frac{\text{vph}}{3{,}600/\text{cycle length}} = \frac{\text{vphg}(g/c)}{3{,}600/\text{cycle length}}$$

Note that the cycle lengths cancel out, leaving

$$m = \frac{\text{green (vphg)}}{3{,}600}$$

where green is in seconds of green time.

This expression is the same as discussed earlier in that the product of green time and vehicles per second of green yields maximum average arrivals per cycle for a given vehicles per hour of green.

With this relationship, points may be plotted on the failure rate chart (Fig. 1) for various numbers of vehicles per hour of green. However, an adjustment must be made to the chart's green time because it contains yellow times and the Manual data do not include yellow times. The authors have assumed a 3-second yellow time for the purpose of this presentation. The Manual states that 2 or 3 seconds' yellow time may normally be expected. However, 3 seconds is generally accepted as a minimum yellow time. The actual suggested computational procedure presented later is such that one may use any yellow time in making level of service checks.

To illustrate the conversion procedure, assume that it is desired to determine the maximum average arrivals per cycle that may be accommodated for a level of service C at a 2-lane through approach. Neither left nor right turns are considered beyond those built into the Manual's charts (the assumption is made that left turns are handled by a separate phase). Also assume that the installation is at an outlying business district location. Data given in Table 1 indicate that no more than 1,200 vehicles per lane per hour of green should approach the intersection to maintain a level of service C. This converts to ⅓ vehicle per second of green. If the actual green time per cycle is 30 seconds, then, in order to plot a point on the failure rate graph, the green time must be multiplied by the vehicles per second of green, which in this example results in an allowable average arrival of 10 vehicles per cycle. A point may then be plotted by determining the intersection with the m arrivals of 10 vehicles per cycle and the green curve of 33 seconds, remembering that the curves include yellow time. This procedure has been followed for the cases given in Table 1, and the various level of service lines have been plotted. Figure 2 shows a plot for a single left turn lane, and Figure 4 shows a plot for a 2-lane approach in a city of 250,000 population and an OBD or fringe area.

However, to this point it has been assumed that the peak-period flow rate equals the peak-hour flow rate, because Figures 2 and 4 are plots of failure rates for the whole peak hour and level of service in terms of vehicles per hour of green. In order to obtain a relationship between peak-period failure rates and levels of service, Drew and Pinnell's equation for determining peak-period factor must be used. As an illustration, each of the m average arrivals plotted in Figures 2 and 4 have been factored up using a peak-period factor obtained from Drew and Pinnell's equation. A city population of 250,000 and an OBD location have been assumed, which is consistent with the Table 1 assumptions. The result of this factoring is shown in Figures 3 and 5. As to be expected, the peak-period failure rate based on a given vehicles per hour of green is greater than the peak-hour failure rate, which according to Drew and Pinnell (3) is not accurately described by the Poisson distribution. As an illustration of the effect that population has on the relation of failure rate and level of service, Figure 6 has been developed to show the varying relationship of failure rate and the lines for the level of service C for populations of 100,000, 250,000, and 1,000,000. The vehicles per hour of green for the different populations were determined using Leisch's MP factor, which will be discussed later. The level of service C is considered significant inasmuch as the Highway Capacity Manual states that this is the level typically associated with urban design practice.

In making actual conversions from failure rate solutions to level of service, the 2 illustrated figures may be consulted if the approach under consideration satisfies the description contained within the figures presented. The procedures would be to enter the graph with the average arrivals per cycle for the peak hour (not peak period) and the computed green time. The area where the intersection of the 2 variables lie would

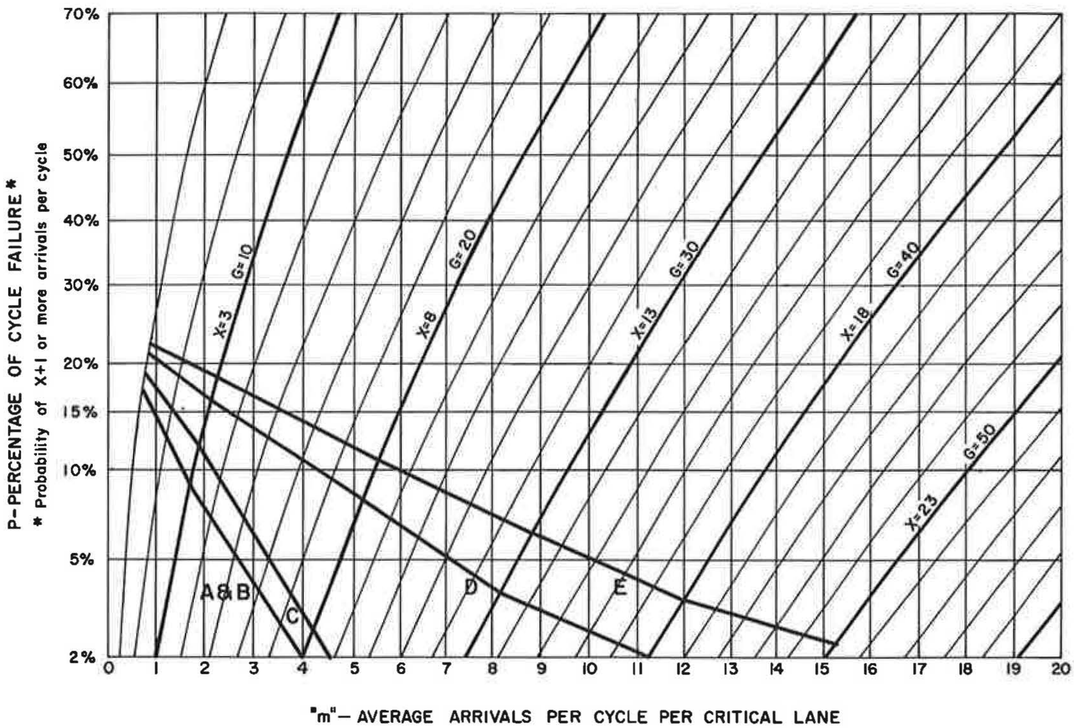

Figure 2. Failure rate and level of service for a single left-turn lane approach (no peak period correction applied).

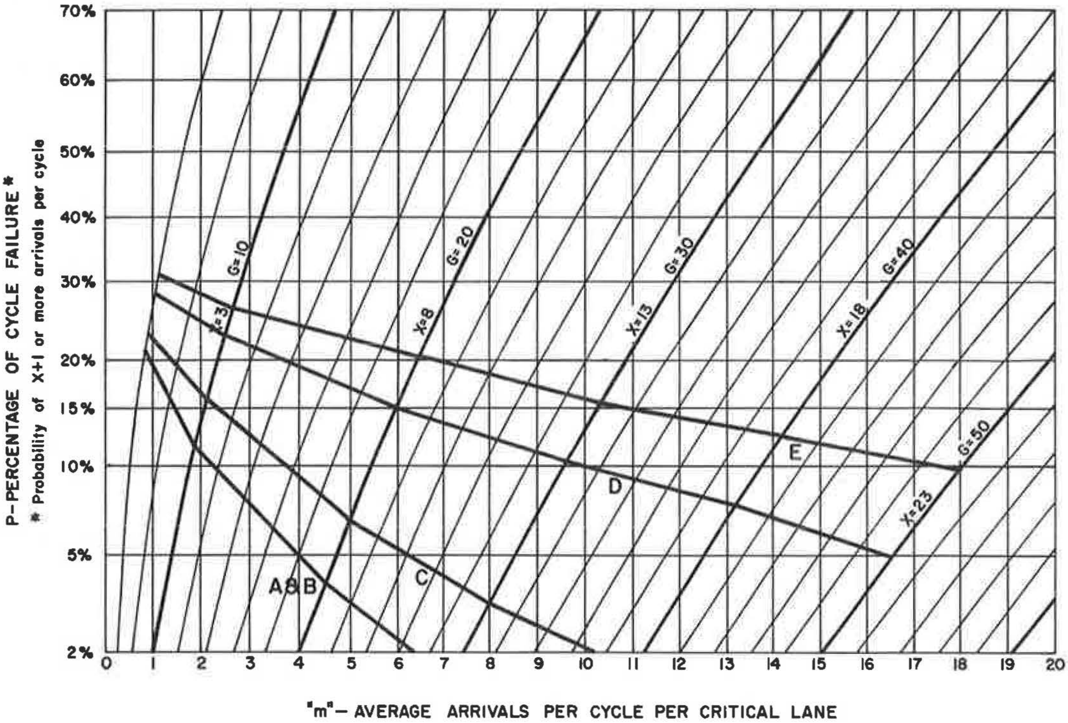

Figure 3. Failure rate and level of service for a single left turn lane approach (peak period correction applied).

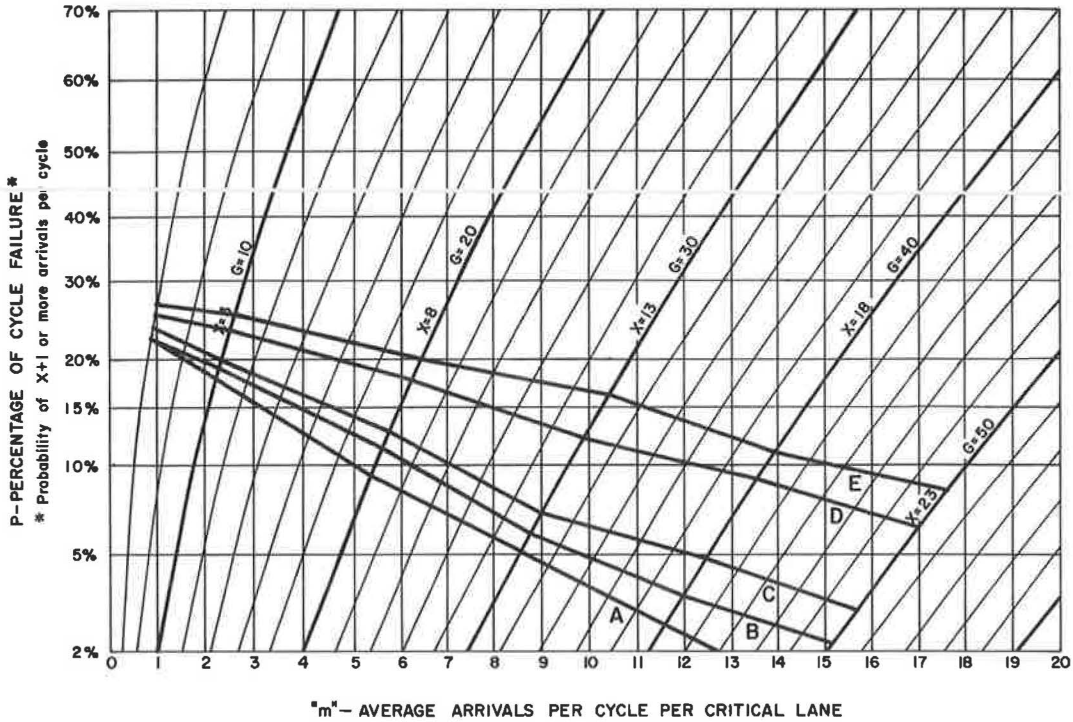

Figure 4. Failure rate and level of service for a two-lane approach, city of 250,000, and an OBD or fringe location (no peak period correction applied).

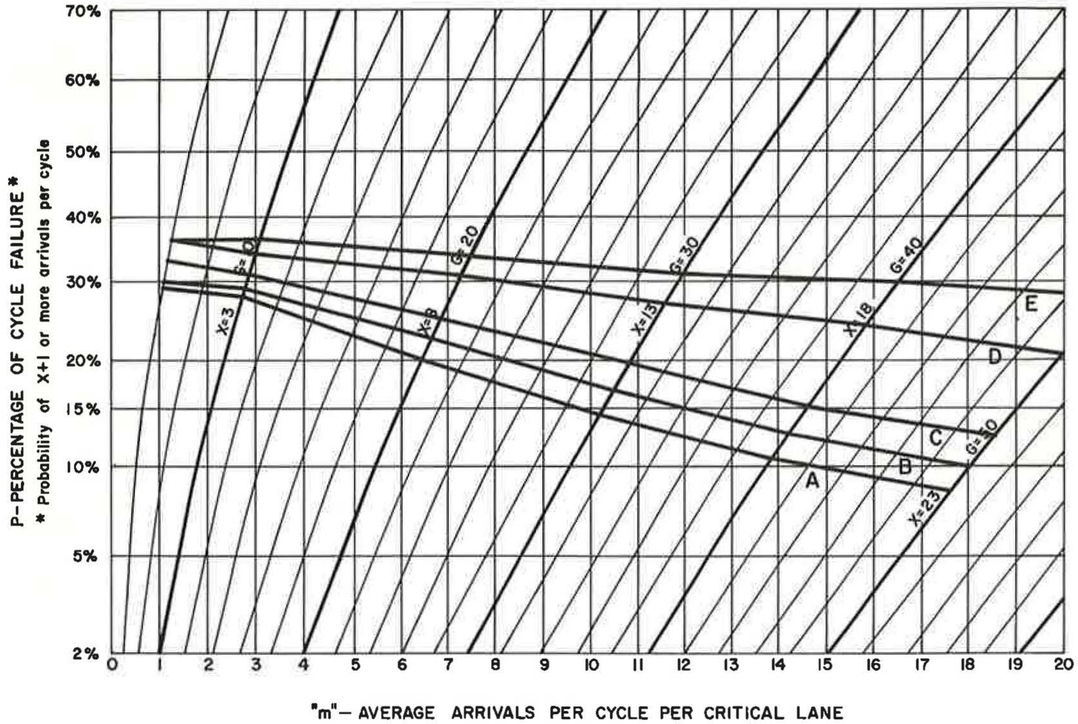

Figure 5. Failure rate and level of service for a two-lane approach, city of 250,000, and an OBD or fringe location (peak period correction applied).

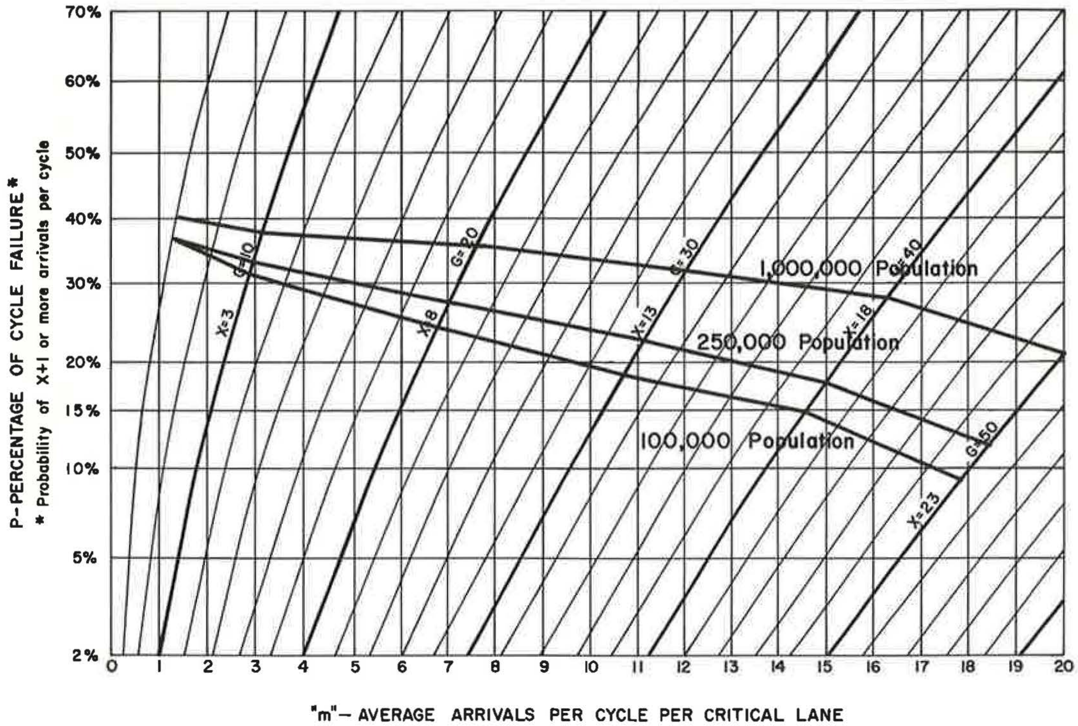

Figure 6. Failure rate and level of service C for different city populations.

indicate the level of service. For example, if the approach under consideration satisfies the description of Figure 4, with an average peak-hour arrival rate of 10 vehicles per cycle and a green time of 30 seconds, a level of service E would be indicated. Note that the intersection is at the 30 percent failure level if a peak-period factor of 1.18 is used commensurate with a city population of 250,000.

Charts could have been prepared for all conditions, but this did not appear warranted. Therefore, an alternate solution procedure would be to take the arrivals per phase per cycle and the green time allotted to each phase and compute the arrival rate in vehicles per second of green (vpsg). Of course, yellow time would be deducted from the green time determined by the failure rate method to determine the arrival rate in vehicles per second of green. It may be noted that Table 1 also gives levels of service in terms of vehicles per second of green. Because Table 1 covers all general cases developed by the Manual, which are considered appropriate for use in solving high-type intersection problems, it may be consulted to determine the level of service for the installation. This table is for a city population of 250,000, and Table 2 should be consulted to adjust the Table 1 values of vpsg, if a city of population other than 250,000 is under consideration. As an illustration, the previous example of 10 arrivals and 30 seconds of green converts to 0.370 vpsg after deducting the assumed 3 seconds' yellow time, which, for use with Table 1, could actually be any reasonable length to be used.

As in the previous example, the description shown in Figure 2 will be assumed, and Table 1 will be used directly, because Figure 2 shows a population of 250,000. The OBD or fringe row and the 2-lane column will be consulted. Because 0.370 vpsg lies between 0.366 and 0.377, a level of service E is again indicated. As stated earlier, if the city population is other than 250,000, the Table 2 factors must be applied. These factors are from Leisch's paper (4). If the actual peak-period factor and population are known, they may be used as shown in Figures 6.5 through 6.9 of the Highway Capacity Manual to obtain an equivalent MP factor for use. To illustrate the adjustment for population, assume that the previous example is used, except that the city population is over 1,000,000. The Table 1 values for a 2-lane approach in a CBD area with appropriate correction factors are given in Table 3. As may be observed, a vpsg of 0.370 is less than the flow rate of 0.380 vpsg, the upper limit for level of service A. Therefore, this design would operate at an A level of service if it were in a city of over 1,000,000 population.

If the failure rate method is to be used by a city traffic engineering department, it is recommended that all data given in Table 1 be converted to the population of the city. Table 1 may then be used as a fast check of the level of service, as defined by the Manual, which would result by designing by the failure rate method. Of course, the average arrivals for the peak hour, not peak period, must be used with this table.

Some conclusions may be drawn from the slope of the level of service lines (Figs. 2 through 6). The slope of these lines indicates that failure rate and level of service, as defined by the Manual, apparently do not correlate. If they were well correlated, one would expect the level of service lines to be relatively level and consistent for different populations. Therefore, it appears that a practical failure rate design level may not be established as suggested by Drew and Pinnell. The acceptable failure rate decreases as the approach volume per cycle increases. Each failure rate solution should therefore be checked by Table 1 to determine the level of service for that solution.

TABLE 2

ADJUSTMENT FACTORS TO BE USED WITH TABLE 1 DATA FOR CITIES OF OTHER THAN 250,000 POPULATION

Population	MP	Population	MP
50,000	0.85	500,000	1.05
100,000	0.90	750,000	1.10
175,000	0.95	1,000,000	1.15
250,000	1.00	over 1,000,000	1.20

TABLE 3

EXAMPLE OF CONVERSION FOR POPULATION

Level of Service	vpsg (Table 1)	1,000,000 Population MP Factor	Adjusted vpsg
A	≤0.317	1.20	≤0.380
B	≤0.324	1.20	≤0.389
C	≤0.333	1.20	≤0.400
D	≤0.366	1.20	≤0.439
E	≤0.377	1.20	≤0.453

AVERAGE INDIVIDUAL DELAY AT A SIGNALIZED INTERSECTION

Some investigators in the field of signalized intersections have recommended that average individual delay be used as an indicator of the level of service offered by a signalized intersection. May and Pratt (5) have expressed some difficulty in correlating load factor with average delay at high load factors. They developed revised level of service load factor limits to obtain a more uniform divergence of average delay for the different levels of service. Their recommendation is given in Table 4.

Because there appeared to be some support for a delay approach to level of service, an investigation was made into the relation that failure rate bears to average individual delay.

To develop this relationship, the delay equation derived by Webster (6) was utilized. Webster's equation is as follows:

$$\bar{d} = \frac{c(1 - \lambda)^2}{2(1 - \lambda x)} + \frac{x^2}{2Q(1 - x)} - 0.65\left(\frac{c}{Q^2}\right)^{1/3} x^{(2 + 5\lambda)}$$

where

$\bar{d}$ = average delay per vehicle on the particular lane passing through the intersection;
c = cycle length;
λ = proportion of the total that is effectively green for the phase under consideration, $\frac{(\text{green - lost time})}{\text{cycle}}$, where lost time is the green time not effectively utilized each phase (Webster recommends a lost time of 2 seconds for the average installation);
Q = lane flow = average number of vehicles passing a given point on the road in the same direction per unit of time;
S = saturation flow = maximum rate of discharge of the queue during the green period; and
x = degree of saturation—ratio of actual flow to maximum flow that can be passed through the intersection on a given lane = $Q/\lambda s$.

An inspection of the variables in the equation indicates that the average individual delay may be computed for each point of intersection of the average arrivals and the green time with its associated maximum departures. A lost time per cycle of 2 seconds was assumed, as recommended by Webster (6). The computations have been made for 40-, 50-, 60-, 80-, and 100-second cycles for a representative number of intersection points.

Knowing the delay at the points where delay was computed, equi-delay lines, corresponding to May and Pratt's recommended delay break points for level of service determination, were plotted. These equi-delay lines were plotted in similar fashion to a contour map for delays of 15, 30, 45, and 60 seconds. Figures 7 and 8 show the equi-delay lines for a 60- and 100-second cycle. Here again some conclusions may be drawn from the slope of the equi-delay lines. Because they are not horizontal, one must conclude that average individual delay apparently does not correlate with failure rate. The acceptable failure rate for a given delay line increases with an increase in volume per cycle.

These 2 charts may be used for checking average individual delay for solutions based on both the failure rate and the level of service methods. Similar charts could easily be developed for a full range of cycle lengths. The delay range may be found by entering the charts with the average arrivals per cycle for the peak hour and finding the intersection with the computed green time and noting where the point of intersection lies with respect to the equi-delay lines.

TABLE 4

MAY AND PRATT'S SUGGESTED BREAK POINTS FOR LEVEL OF SERVICE (5)

Level of Service	Revised Load Factor Limits	Average Individual Delay (seconds per vehicle)
A	≤0.1	≤15
B	≤0.58-0.66	≤30
C	≤0.66-0.82	≤45
D	≤0.72-0.91	≤60
E	≤1.0	>60

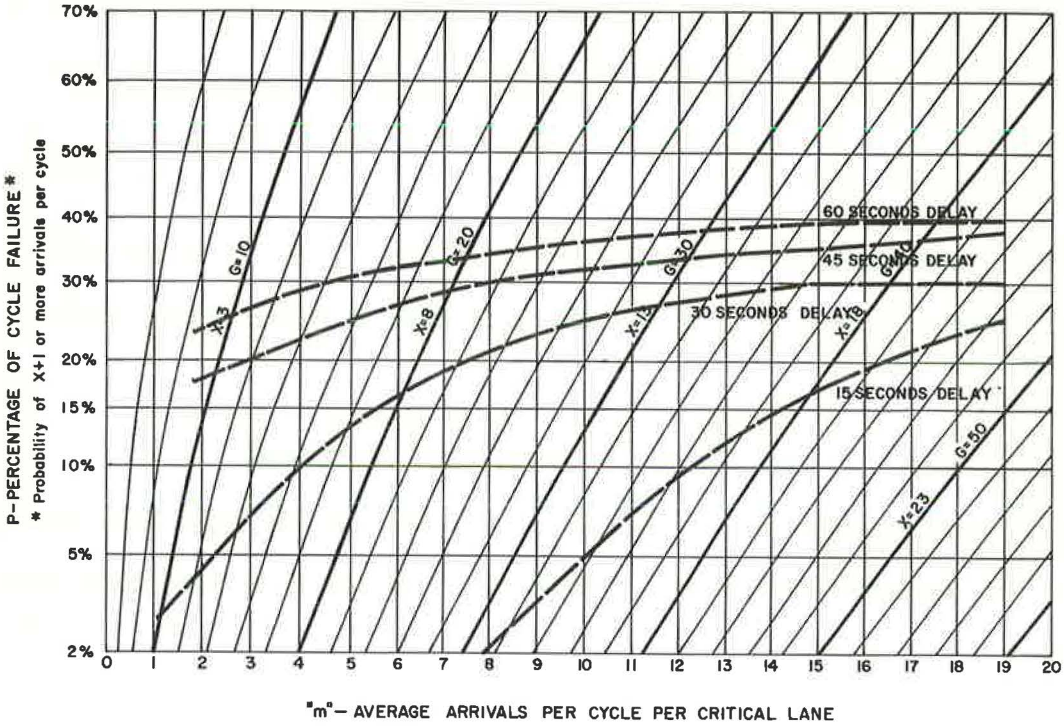

Figure 7. Failure rate and average individual delay in seconds per vehicle for a 60-second cycle.

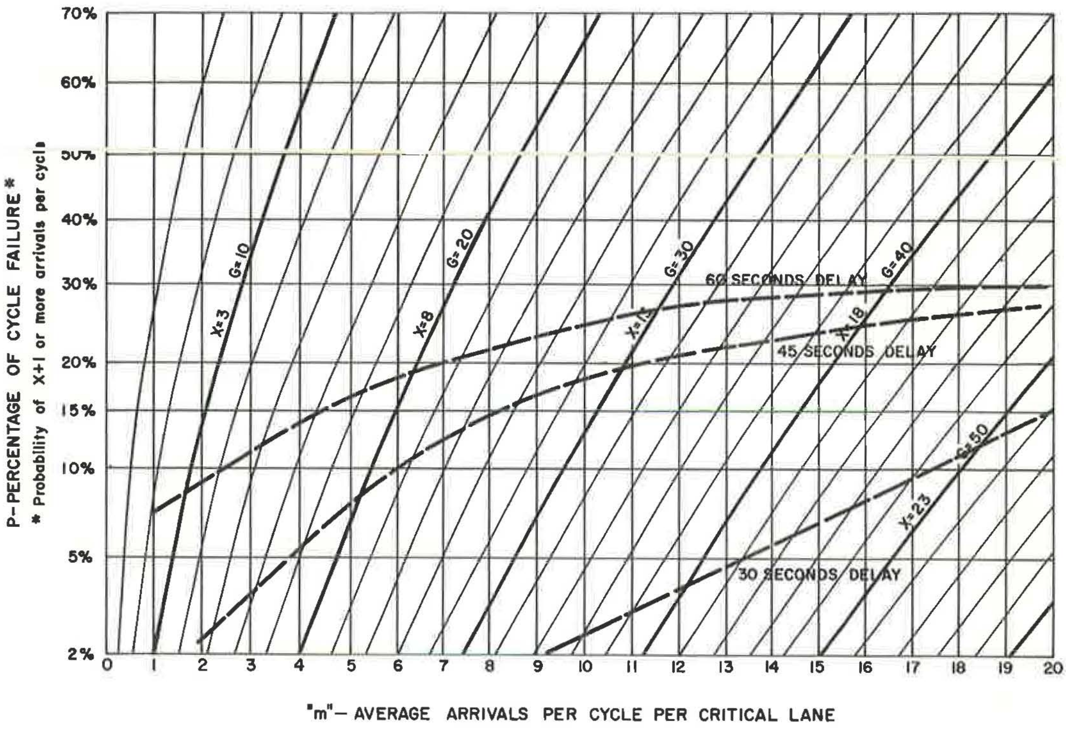

Figure 8. Failure rate and average individual delay in seconds per vehicle for a 100-second cycle.

RELATION OF LEVEL OF SERVICE TO AVERAGE INDIVIDUAL DELAY

A general inspection of the slopes of the lines for the 2 graphical relationships, developed to this point, raises some question as to the relationship that level of service, as defined by the Manual, actually bears to average individual delay.

Figure 9 shows delay lines for a 60-second cycle (Fig. 7) and the level of service lines for the 2-lane approach (Fig. 4). It is readily apparent that the level of service lines cut across all delay lines indicating delays from less than 15 seconds to over 60 seconds. This indicates that 2 signalized intersections may satisfy the Manual's criteria of vehicles per hour of green for a given level of service, but the intersections may have considerable difference in average individual delays, depending on the volume being accommodated per cycle. An inspection of the other delay and level of service charts indicates that similar relationships exist for other cycle lengths and approach configurations.

This deviation is due in part to the fact that the Manual's criteria of vehicles per hour of green does not take into account cycle length or g/c ratios. Cycle length is well known for having an effect on average delay; this subject has been investigated by several individuals including Webster (6). The g/c ratio has an effect on delay due to the different percentages of the green time that are not utilized. A design start up and clearance time is stated by Drew and Pinnell (3) to be 6 seconds per phase. The fact that the Manual does not include yellow time in its green designation does not offset the loss of 6 seconds per cycle. Therefore, as green times get shorter, there is an inequity in the relationship that green time less yellow time bears to usable green time (green time less start up and clearance time).

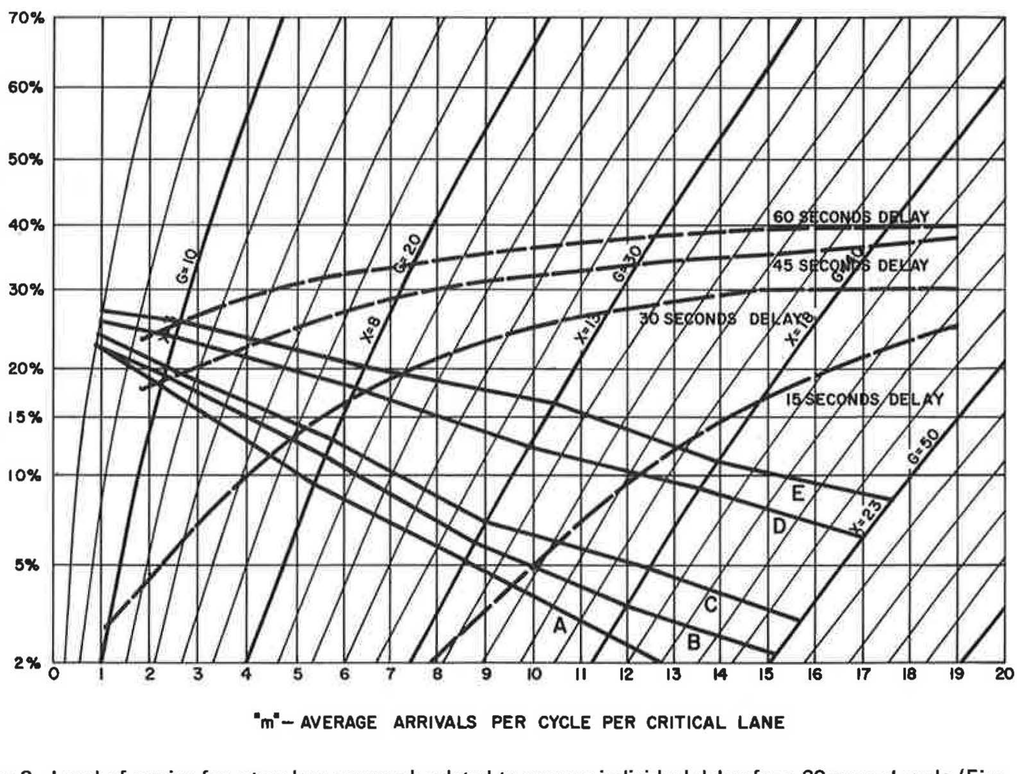

Figure 9. Level of service for a two-lane approach related to average individual delay for a 60-second cycle (Figs. 4 and 7 combined).

CONCLUSIONS AND RECOMMENDATIONS

When using the failure rate method of intersection design, one should check the level of service actually provided. Delay checks should also be made for each phase. The tables and charts presented here could be of some use in making these checks. For use in a particular city, Table 1 should be adjusted for that city's population.

It is recommended that consideration be given to including average individual delay as an index of the level of service offered by a signalized intersection. If speed is to be considered the criterion for uninterrupted flow conditions, then a delay index appears commensurate for intersection design. Perhaps one of the objections to using delay is the difficulty in obtaining field data for average individual delay. No doubt load factor is considered an easier field measurement. However, Sagi and Campbell (7) have developed an equation for determining average individual delay that does not require any more field work than a load factor determination. Perhaps this may remove some objections to the use of average individual delay as an index of the level of service at an intersection. If delay is to be accepted as an index, the Manual's present term of vehicles per hour of green will have to be modified to account for cycle length and g/c ratios. An inspection of Figure 9 makes this need apparent.

If average individual delay is to be used extensively for design, it is recommended that a nomogram be developed to facilitate the use of Webster's equation. Using such a nomogram, actual saturation flow rates and start up and clearance times could be used for a particular intersection and average individual delays easily obtained.

ACKNOWLEDGMENT

This paper is based on Mr. Tidwell's thesis, The Relation of Failure Rate at a Fixed Time Signalized Highway Intersection to Level of Service as Defined by the 1965 Highway Capacity Manual, presented to the University of Tennessee, September 1969. The assistance of W. H. Wilson and members of the Research and Planning Division of the Tennessee State Highway Department in the preparation of the illustrations is greatly appreciated.

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Discussion

G. W. SKILES, Los Angeles Department of Traffic—One view of the research reported on here is that 2 alternative approaches to the capacity analysis of signalized intersections yield results that are inconsistent and that neither approach produces solutions that correlate with average delay, a quantity thought by some to be a desirable figure of merit.

Taking this view, the results of the research are disappointing. There is not much advantage in having alternative approaches if they will not give the same answer. It is worse if neither answer is correct.

However, the purpose of the research was not to evaluate the 2 (or 3) alternate methods of capacity analysis. It was to develop a methodology whereby a solution obtained by using the failure rate method of design could be expressed in terms of expected level of service and delay. This was done. The results could be of significant value to those who wish to use the failure rate technique.

In addition to allowing a solution to be expressed in terms that may be more meaningful, the authors' results could provide a very useful aid in the use of the failure rate design procedure. As Drew and Pinnell point out, in using their procedure one has an infinite combination of phase lengths from which to choose. The same failure rate need not be used for all phases.

The authors' methodology provides the designer with a tool for limiting his appropriate field of choice. If, for example, he wishes to provide the same level of service for opposing phases, the failure rate-level of service comparison allows him to do so quite easily. Similarly, the chart showing both failure rate and delay facilitates a selection of phase length combinations for equal average delay or for any desired ratio of delays.

The unfortunate part of the authors' results is that one is still faced with an apparently conflicting choice. His selection of appropriate phase lengths will differ, depending on whether he wishes to use delay or level of service as a criterion. This may or may not be logical or desirable.

The authors' implication is that the element in error is the level of service criterion. I am less certain that this is the case. The final figure shows failure rate, level of service, and average delay superimposed. An examination of the chart for logical relationships does not lead to firm conclusions. However, there are some such relationships.

Comparing failure rate and level of service, it is found that, for a given approach volume, as failure rate increases, the level of service decreases—a result one should expect. Why the level of service curves should not be more nearly parallel to the failure rate lines, though, I do not understand. If I do understand the procedure followed, the level of service curves (expressed in terms of service volume) are derived basically from the load factor curves in the Highway Capacity Manual. By definition, load factor and failure rate are very similar. Miller (8) indicates that they are related approximately by the ratio $e^{1.58\phi}/e^{1.30\phi}$, where ϕ is a function of flow rate, saturation flow rate, and degree of saturation.

I note, incidentally, that the level of service curves as plotted on the failure rate charts show, for a given level of service, a nearly constant ratio of $m/(x + 1)$, where m = average volume per cycle and $x + 1$ = the lowest volume constituting a cycle failure. I wonder if this has a pertinent meaning.

A rationale that could be developed from the failure rate-level of service comparison goes like this:

> At lower volumes, a higher failure rate can be accepted for a given level of service because, upon failure, there are fewer vehicles in queue and green intervals occur frequently. The queue left over can be expected to clear the next cycle. A cycle failure will occur only 20 percent of the time, at most, if one limits his choice to the A through E level of service area.
>
> At higher volumes, a cycle failure is more serious. The number left in queue is likely to be higher than for the previous case. Hence, for a given level of service, one should use a lower failure rate for higher volume levels.

The failure rate-average delay comparisons indicate an opposite conclusion. After all, the reason one would be concerned about the number of times a queue remains, and the number left in queue, is because of the effect on delay. As the authors point out, though, the results indicate that a higher failure rate can be accepted at higher volume levels for constant delay. To me, this is not a logical relationship. I question that it should be so, although my main objection may be that it casts a shadow on my earlier rationalization.

The failure rate-delay comparisons do show some logical consistencies. For a given cycle length and green interval, delay and failure rate increase as average approach volume increases. For a constant cycle length and volume, delay and failure rate increase with decreasing green time. For a given green interval and average arrival rate (g, m, and x constant), delay increases with increasing cycle length; failure rate is constant.

These relationships are all to be expected, of course, and, so far, I do not find justification for my uneasiness. However, the last relationship, especially, may illustrate something. In that case, failure rate is constant, even though cycle length and delay are varying. In other words, delay is not a function of failure rate. Failure rate is not a function of cycle length. Both failure rate and delay are functions of green time and arrival rate; delay is, in addition, a function of cycle length. It may be incorrect to compare the 2 quantities, delay and failure rate, in the way that was done.

A point noted from the final chart is that the degree of saturation (in Webster's equation) exceeds unity above a line roughly approximating the 45-second delay line on the 60-second cycle chart. If the delay relationship is correct and the level of service relationship is incorrect, this would indicate that the upper limit of level of service E should be at about that same point. Perhaps the level of service curves, then, should have slopes nearly the same as those for delay. This might be a starting point for revising the level of service curves.

Another point brought out by the final chart is, I think, much more interesting. That is that product $\bar{d}$ m is nearly constant for a given level of service. In other words, for the conditions of that chart, there is a close correspondence between level of service and total delay (not average individual delay). If this relationship is consistent for other conditions, the result could be extremely meaningful.

I conclude that I have no firm views on the apparent inconsistencies in the authors' final result, except that, possibly, the most disturbing inconsistency is eliminated if one uses total delay rather than average individual delay. I do have the observation that, while delay is an appealing figure of merit, the relationship between service volume and delay is often an erratic one. Normann (9) pointed this out and gave this as one reason for selecting load factor, rather than delay, as a criterion for signalized intersection capacity. Some of our studies have shown similar inconsistencies between service volumes and load factor. May and Pratt's study, referred to by the authors, does not show a consistent relation between load factor and delay except at very low load factor levels. We seem to have need for additional facts.

In reviewing the paper, a question keeps arising: Should we really be surprised if figures of merit developed from bases involving different sets of assumptions fail to agree? Perhaps we should be more surprised if they did agree.

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JAMES H. LITTLE, Missouri State Highway Commission—It is becoming increasingly more apparent that the search for the most acceptable method of determining intersection approach capacities must continue. The refined computation procedure described in the 1965 Highway Capacity Manual is unquestionably a step forward; however, it is not without its weaknesses. Chang and Berry have discovered apparent discrepancies between some of the Highway Capacity Manual curves, and, as pointed out by the authors of this paper, May and Pratt have found inconsistencies in the ranges of average individual delay associated with the various load factor groups. A study comparing estimated and observed service volumes of 90 signalized intersection approaches in Missouri's 3 largest cities has indicated average errors of 47.7, 32.0, 16.1, and -4.5 percent for service levels B, C, D, and E respectively. Clearly, further refinements of the data and procedures contained in Chapter 6 of the Manual are needed.

The authors of this paper have brought us another step nearer the better solution we seek. By a rather ingenious application of the failure rate design chart developed by Drew and Pinnell, they have demonstrated that the failure rate design method and the current level of service design method are not well correlated. As the authors point out, we would expect the level of service lines of Figures 2 and 4 to be relatively level if good correlation exists. An examination of their charts shows that this would be possible only if the Manual's vehicles per hour of green for a given level of service were allowed to vary with the allotted green time.

In Figures 3 and 5, the authors show that better correlation exists between the 2 design methods when the flow rates for the various service levels are adjusted to represent peak-period rather than peak-hour rates. In fact, Figure 5 shows that the correlation between the 2 methods is reasonably good if a 2-lane approach is designed for adjusted capacity (level E) flow rates.

In the latter part of their paper, the authors conclude that deviations in the relationship between average individual delay and level of service curves are due in part to the fact that the Manual's criteria of vehicles per hour of green does not take into account cycle length or g/c ratios. This shortcoming may also explain why the failure rate and level of service curves are not better correlated.

Inasmuch as the authors conclude their paper by recommending that consideration be given to including average individual delay as an index of the level of service and modifying the Manual's present term of vehicles per hour of green to account for cycle length and g/c ratios, their earlier conclusion that it may not be possible to establish a practical failure rate design level may be premature. For the present, the failure rate method at least provides a good check of the adequacy of a level of service design.

One limitation of the failure rate design method developed by Drew and Pinnell is that the equation for estimating peak flow rate within the peak hour does not apply to the larger metropolitan areas. If applied to a metropolitan area of 1,667,000 or more population, it will indicate a peak-period flow rate less than the average peak-hour flow rate. Wherever possible, actual counts at the site in question, or average peak-hour factors, should be used to determine peak flow rates.

The table developed by the authors (Table 1) for estimating the level of service provided by a given approach, when the lane layout and signal phasing are known, might be useful for a rough check of level of service on approaches similar to those covered by the table; however, because of the many possible combinations of lane width, percent turns, and the like, it is felt that generally it would be better to use the Manual's charts, or the nomographs developed by Leisch, in making such checks.

The authors' use of average individual delay curves corresponding to the load factor break points suggested by May and Pratt was of considerable interest to those of us who worked on the previously mentioned Missouri study because we feel these break points are more realistic than those presently in use.

The authors' finding that average individual delay does not appear to be correlated with level of service, as presently defined by the Manual, is noteworthy. Their suggestion that the correlation might be improved by making the Manual's criteria of vehicles per hour of green more responsive to the effects of cycle length and g/c ratios deserves serious consideration.

Messrs. Tidwell and Humphreys have made a significant contribution to the store of knowledge concerning signalized intersection capacity and should be congratulated on the result of their efforts.

DAVID SOLOMON, U. S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads—The authors have certainly presented a very useful paper. Their general finding, that there is no correlation between level of service, failure rate, and average individual delay, suggests that it would be useful to investigate at a more fundamental level the basic criteria employed in evaluating intersection performance.

The basic criteria used in these analyses are delay, stops, and travel speed or time. The question is, How do drivers evaluate these criteria? For example, is one minute of delay and one stop less desirable than 20 seconds of delay and 2 stops? Once a better understanding has been obtained of these relationships, it will be possible to design signal timing schemes based on criteria that are important to drivers.

The next questions is, How should research on the desires of drivers be carried out? A number of techniques might be explored. Direct questions might be asked, or a more advanced type interview technique could be employed involving development of an attitude scale.

An experimental approach might be tried in a laboratory. A group of test subjects could be shown films of several traffic situations and asked to evaluate delay and stops in terms of a subjective scale or in terms of the cost they would assign to each level of delay or number of stops.

Field experiments could be tried by giving drivers alternate route assignments, having them evaluate the routes subjectively or in terms of cost, and correlating with the stops and delays. A refinement of this could involve giving test subjects a certain sum of money and requiring them to pay back some of it in return for reduced stops or delays. This could be employed in either a laboratory-type situation or on the street, with car pools, for example.

JOHN E. TIDWELL, JR. and J. B. HUMPHREYS, Closure—The authors would like to thank the Highway Research Board for the opportunity of bringing their findings to the attention of the profession. Appreciation is also expressed to Messrs. Skiles, Little, and Solomon for their discussion comments. Their comments are very appropriate and should be of assistance in the further exploration of this topic.

By way of specific comment, Mr. Skiles' suggestion that total delay and the level of service lines may be correlated is not borne out when other cycle lengths and populations are taken into consideration. Admittedly, the correlation is greatly improved over the average individual delay-level of service relationships. The writers realize that total delay may be a useful index of signal efficiency, but, because total delay intimates that average individual delays may vary depending on average arrivals, we do not recommend it for a level of service index. Miller's work (8) regarding a load factor equation has been followed up. A plot of the 0.1, 0.3, 0.7 load factor lines on the failure rate chart yields peak-hour failure rates of approximately 7, 18, and 32 percent. This compares closely with a simulation study by the writer for Poisson arrivals. A 1.00 load factor was obtained at the 55 percent level. This still does not give a relation of peak-period failure rate to the peak-hour load factor. Additional research in this area should prove useful.

We can find no meaningful explanation for Mr. Skiles' $m/(x + 1)$ relationship. Following Mr. Skiles' lead, a rationale for the failure rate-delay conclusion is as follows: Two signals may have the same failure rate with one having low arrivals and low green time and the other higher arrivals and longer green time. The signal with the short green obviously must have a longer red phase than the one with the long green time. Therefore, any overflows from a previous cycle or arrivals during a red phase must wait longer in the queue than would be necessary for the signal with a long green phase.

If this paper has generated meaningful discussion, which may lead to "the better solution" referred to by Mr. Little, then the time and effort expended in the preparation of this paper have been very worthwhile.

Freeway Travel Time Evaluation Technique

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•THIS REPORT describes a systematic analytical procedure to evaluate the effectiveness of freeway improvement plans such as design modifications and ramp control implementations. In this analytical procedure, the total travel time along the freeway under given physical and traffic conditions is calculated systematically. The procedure consists of 2 submodels: freeway capacity analysis and freeway travel time calculation. The capacity analysis for freeway, ramps, and weaving sections is carried out according to the 1965 Highway Capacity Manual. Demands are calculated from origin and destination (O-D) tables of 15-minute time intervals, and the travel speed for each freeway subsection is estimated using the relationship between volume to capacity (v/c) ratio and operating speed. The travel time is calculated for both nonqueuing and queuing situations. In addition to the theoretical development and the structure of the model, computer operation procedures such as input coding system and output format are included, and the results of sample outputs are compared with the data collected from the East Shore Freeway.

MODEL STRUCTURE

Description of Systems

Physical Condition of Freeway—To make a reasonable estimation of travel time on a freeway, it is necessary to know the physical and operational characteristics of the freeway and to put them into an appropriate numerical expression.

In general, freeway sections exhibit a number of varying design and operational features. Thus, to establish a meaningful relationship between traffic volume and the average speed of traffic, it becomes necessary to divide the freeway section into several homogeneous subsections that exhibit the properties of constant capacity and demand over their lengths. It is also necessary to itemize the features that affect the capacity of each subsection such as design speed, number of lanes, lane width, percentage of grade, grade length, and location of on- and off-ramps. Traffic factors that affect subsection capacities and that are hypothesized to be constant over the peak period should also be listed in the same table. It is convenient for later analysis to list all these elements in the format shown in Figure 1.

Traffic Demand—Traffic demands are introduced into the freeway section in the form of O-D tables. The first subsection and each on-ramp are considered as origins, and each off-ramp and the last subsection (NSEC) as destinations. The origins and the destinations are numbered consecutively from upstream to downstream as shown in Figure 1.

Considering the fact that traffic demands during a peak period usually vary, the peak period should be divided into a number of smaller time intervals. It is therefore necessary to input an O-D table for each time interval.

This method for treating traffic demand, although adding complexity, yields the following desirable characteristics: (a) Actual demand patterns are more realistically simulated; (b) travel times for individual O-D movements can be readily obtained and

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SUBSECTION NUMBER	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
LENGTH IN FEET	1630	1960	1550	1960	300	4790	3030	2160	2030	1250	900	1320	720	2610	1660	1090	2310	1460	3800	1100	660	1440	1480	600	4690	2190	2320	830	1180	2560
NUMBER OF LANES	5			4								3																		
VOLUME COUNTS	601-602-603-605-607 — 608 — 609 — 610 — 611 — 612 — 613 — 614 — 615 — 616 — 617 — 618 — 619 — 620 — 621 — 622 623 — 624 625 — 626 627 628 — 629																													
PEAK HOUR FACTOR	0.91																													
LANE WIDTH	12.0																													
OBSTRUCTION DIST.	6	6	6	6		6	6	6	6	6	6	4	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
% OF GRADE	0	0	0	0		0	0	0	0	0	0	0	0	0	0	0	2	0	0	2	0	0	0	0	0	0	0	3	4	4
LENGTH OF GRADE	–	–	–	–		–	–	–	–	–	–	–	–	–	–	–	2310 (0.44)	–	–	1100 (.208)	–	–	–	–	–	–	–	1280 (.24)	2440 (.46)	5000 (.94)
% OF TRUCKS	6	4	4	5		4	4	4	4	3	5	4	4	3	3	3	3	2	3	3	3	3	2	2	4	3	4	2	2	4
AVG. HIGHWAY SPEED	70.0																													

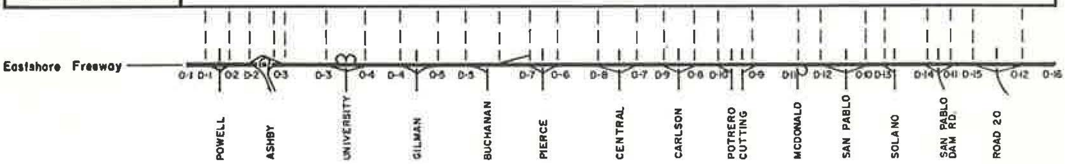

Figure 1. Freeway subsection parameters.

are essential for evaluating the effectiveness of such improvements as ramp control; and (c) the resultant freeway model exhibits a flexibility that will facilitate considerations of network traffic movements and patterns.

Basic Assumptions—Following are the basic assumptions of the model:

1. Traffic is treated as a compressible fluid where an individual vehicle is regarded as an integral part of the fluid and is not considered individually.
2. Within a given time interval, traffic flows remain constant and do not fluctuate over that time interval. For a given subsection, traffic demands are expressed as a step function over the entire time period under consideration.
3. Capacities of subsections, including weaving sections and merging points, are estimated using Highway Capacity Manual (1) methods.

Model Development

Demand Calculation—As mentioned previously, an O-D table format is used to input traffic demand. It is therefore necessary to calculate total demands for on- or off-ramps and for each subsection from the O-D table.

Let NTRIP (I, J) be the traffic demand (number of trips) between the I th origin and the J th destination:

$$\mathrm{NTOR}\ (I) = \sum_{j} \mathrm{NTRIP}\ (I, j)$$

$$\mathrm{NTDE}\ (J) = \sum_{i} \mathrm{NTRIP}\ (i, J)$$

where NTOR (I) is the I th on-ramp demand and NTDE (J) is the J th off-ramp demand.

From assumption 2, the subsection demand between the I th origin and the J th destination can be calculated as follows:

$$\mathrm{IVOL}\ (L) = \sum_{i=1}^{I} \mathrm{NTOR}\ (i) - \sum_{j=1}^{J-1} \mathrm{NTDE}\ (j)$$

where L is any subsection number between the I th on-ramp and the J th off-ramp.

Capacity Analysis—The Institute of Transportation and Traffic Engineering has already developed a series of computer programs for capacity analysis in accordance with the 1965 Highway Capacity Manual. Therefore the main efforts of this study were directed toward fitting these programs into the system of the freeway model.

Freeway Capacity—The basic relationship of the freeway capacity program is

$$SV = 2{,}000NWT(v/c)$$

where

- SV = service volume in vehicles per hour,
- N = number of directional lanes,
- W = adjustment factor for lane width and lateral clearance,
- T = truck factor, and
- v/c = volume to capacity ratio.

The program (freeway capacity submodel) is an independent submodel, and the outputs of this submodel, such as freeway capacity and truck factor, become inputs for the main routine for the freeway model. Special consideration is required for those subsections with an auxiliary lane or the subsections where the number of lanes change, because the freeway capacity submodel cannot handle capacity analysis for subsections with special geometric features.

Ramp Capacity—Because the freeway model is supposed to be used to analyze traffic flows on a critical bottleneck section in a freeway system, the ramp capacity analysis is based on the D-E method, which is used for level of service D or E.

In this ramp capacity analysis, the lane 1 volumes at merging points (500 ft downstream of each on-ramp) are compared with the merging capacity, which is 2,000 vehicles per hour. If the lane 1 volume at the merging point is found to be greater than the merging capacity, the exceeded demand is stored at the merging point and the queue length and the delay time caused by the merging restriction are computed following usual queuing theory.

Several check systems are included in the ramp capacity analysis routine in order to supplement the merging volume analysis, which is based on the D-E method; therefore, the routine cannot handle ramp capacity analysis for unusual ramp design features such as left-side ramps and the 2-lane ramps. There are 3 kinds of input data for this check system: special ramp indicator, on-ramp limit, and off-ramp limit. If there are any left-side ramps or 2-lane ramps, the special ramp indicator should be coded 1 or 2 respectively, and the capacities of those ramps may be input to the computer in the form of ramp limits. The ramp limits, in this case, should be the best estimation for capacities of those special ramps. Traffic demands at special on-ramps are compared with ramp limits, and, if the demand exceeds the ramp limit at the special on-ramp, the delay time and queue length are computed and the results are printed in the computer output.

Ramp limits are set to 1,500 vph for usual ramps. By reducing this value to the ramp metering rate, it is possible to evaluate the effect of ramp control plans on freeway traffic.

Off-ramp demands are merely compared with ramp limits. If the demands exceed the ramp limits at certain off-ramps, a statement is printed in the computer output just to show the excess demands at those off-ramps. There are no particular computation procedures for the queue length or the delay time evaluation for the off-ramp excess demands.

Weaving Capacity—For a given weaving section, the length of the weaving section can be found from the subsection parameter table, and the weaving volumes can be calculated from the O-D tables for each time interval. Then the value of the weaving influence factor K can be found using Figure 7.4 of the Highway Capacity Manual. Then if K is greater than one, the maximum volumes for the designated levels of service can be calculated using the following formula given in the Highway Capacity Manual:

$$SV = \frac{V + (K - 1) \times W_2}{N}$$

where

V = total volume in vph,

N = number of directional lanes,

SV = service volume in vph/lane, and

W_2 = smaller weaving volume in vph.

Taking the upper limit of level of service E,

$$N \times SV_E = C$$

Then, adjusted capacity C′ would be

$$C' = C - (K - 1) \times W_2 \tag{1}$$

The first half of the weaving capacity subroutine is directed toward finding the location of ramps, the length between ramps, and configuration of ramps, using weaving volumes derived from O-D tables. A maximum of 2 adjacent sections can be analyzed in case of multiple weaving.

Estimation of Average Speed—The average speed of each subsection is estimated from the relationship between the v/c ratio and the operating speed shown in the Highway Capacity Manual. In Figure 9.1 in the Manual, the operating speed is expressed as a function of v/c ratio, number of lanes in one direction, and design speed of the freeway.

For convenience of computer operation, all the curves in Figure 9.1 are fitted by various polynomials. The operating speed is then converted to the average speed by

$$V_A = V_O - DS/10 \times (1 - v/c)$$

where V_O is the operating speed in mph and DS is the design speed of the freeway in mph.

Travel Time Calculations—As long as the capacity is greater than the demand, the travel time for a given subsection and a given time period can be calculated as follows:

$$TT = \frac{L}{5{,}280 \times V_A} \times D \times T_O \tag{2}$$

where

TT = travel time in vehicle hours,

V_A = average speed in miles per hour,

D = demand for a given time period in hourly rate of flow,

L = length of subsection in feet, and

T_O = time interval;

while

$$D = V_A \times d \tag{3}$$

where

d = density in vehicles per mile,

then

$$TT = \frac{d \times L}{5{,}280} T_O$$

Queuing Extension of Freeway Model—When demands exceed capacities for certain subsections, physical queues occur in the upstream of these bottleneck subsections.

If the density of traffic flow in a queuing situation is known, it is possible to estimate the physical queue length using the following equations:

$$NP_i = D_i - C_i \tag{4}$$

and also

$$T_O \times NP_i = H_{i-1} (d'_{i-1} - d_{i-1}) \quad (5)$$

where

NP_i = rate of flow of excess demand at subsection i in vph,
D_i = demand for the bottleneck subsection i in vph,
C_i = capacity of subsection i in vph,
H_{i-1} = length of the physical queue formed upstream of subsection i in miles,
d'_{i-1} = queuing density in subsection i-1 in vph, and
d_{i-1} = nonqueuing density in subsection i-1 in vpm.

Then, if the travel speed in queuing density is known, it is possible to evaluate the delay time caused by the bottleneck and the effect of the bottleneck on the total travel time.

Densities and travel speeds in queuing situations can be estimated by using the relationship between v/c ratio and average speed as it was done for nonqueuing situations. If, for a certain time interval, the demand exceeds the capacity in subsection i, the flow rate in subsection i should be equal to the rate of capacity flow; and the average speed of traffic in capacity flow should have the value corresponding to v/c = 1 on the curves in Figure 9.1 of the Highway Capacity Manual. The demands of downstream subsections should be recalculated based on the capacity flow rate of subsection i.

The traffic volume upstream of subsection i, for example for subsection i-1, should be

$$U_{i-1} = D_{i-1} - NP_i$$

where

D_{i-1} = demand for subsection i-1 at this time interval, and
U_{i-1} = volume of traffic leaving subsection i-1.

Then if $D_{i-1} < C_{i-1}$, the travel speed in subsection i-1 can be estimated by reading the value of the speed corresponding to $v/c = (D_{i-1} - NP_i)/C_{i-1}$ on the dotted line for level F in Figure 9.1 of the Highway Capacity Manual. Then the queuing density for subsection i-1 can be calculated from Eq. 3, and the physical queue length at the end of this time interval can be calculated from Eq. 5.

If the physical queue length exceeds the length of subsection i-1, the physical queue length extended into further upstream subsections should be considered in the same way.

Then travel time for subsection i-1 can be expressed as follows:

$$TT = t \times d_{i-1} \times L_{i-1} + (d'_{i-1} - d_{i-1}) \frac{1}{2} t^2 r + (T_O - t) d'_{i-1} \times L_{i-1}$$

where

$$r = \frac{NP_i}{d'_{i-1} - d_{i-1}},$$

$$t = \frac{L_{i-1}}{r},$$

L_{i-1} = length of subsection i-1, and
T = time interval, 0.25 for 15-minute interval.

If $D_{i-1} > C_{i-1}$, the excess demand of subsection i-1 is added to NP_i, and the computer proceeds to subsections further upstream following the procedure described earlier until the computer finds a nonsaturated subsection.

When the demand becomes less than the capacity at subsection i, but physical queues still remain in upstream subsections i-1, i-2,..., stored vehicles are discharged into downstream subsections through subsection i with the rate of NP_i, where $NP_i = C_i - D_i$.

The travel speed in queuing densities, the decrease of the physical queue length, and the travel time can be calculated using a similar method described earlier.

TABLE 1

DATA CARD CODING FORMAT FOR FREEWAY CAPACITY ANALYSIS

Field	Description	Columns	Field Format
1	Title	1 through 28	Flexible
2	Number of lanes	30	1I
3	Peak-hour factor	34 through 37	3F.2 (0.77 to 1.00)
4	Lane width	39 through 41	2I (9, 10, 11, 12)
5	Obstruction distance to right	43 through 44	1F (0. to 9.)
6	Obstruction distance to left	46 through 47	1F (0. to 9.)
7	Percentage grade	49 through 50	1F (0. to 7.)
8	Length of grade	52 through 55	3F.2 (0, 0.25 to 4.00)
9	Percentage trucks	57 through 59	2F (00. to 20.)
10	Average highway speed	61 through 63	2F (50, 60, 70)

Note: One card for each subsection.

PROCEDURE FOR COMPUTER CALCULATION

Freeway Capacity Submodel

The data card coding format for the freeway submodel program is given in Table 1. Data cards should be prepared for each freeway subsection.

When data cards are placed in the computer, a number of checks are made prior to the actual capacity calculations.

After the initial checks have been completed and adjustments in input data made, the program completes the capacity calculations. The 3 major portions of the output are input, termination or modification statements, if any, and results.

Freeway Travel Time Submodel

There are 4 different formats for the model input: control card, freeway subsection parameters, on-ramp volume limits, and O-D tables. The coding formats for the freeway subsection parameters and O-D tables are given in Table 2.

TABLE 2

DATA CARD CODING FORMAT FOR TRAVEL TIME COMPUTATION

Field	Description	Columns	Field Format
	Freeway Subsection Parameter[a]		
1	Subsection number	4 and 5	2I
2	Number of lanes	10	1I
3	Capacity, vph	11 through 15	5I
4	Subsection length, ft	16 through 20	5I
5	Truck factor	21 through 25	4F.3
6	On-ramp indicator	29	O, if there is an on-ramp
7	Off-ramp indicator	30	D, if there is an off-ramp
8	Special ramp indicator	34	1 for left-side ramp and 2 for 2-lane ramp
9	Remarks	36 through 80	Flexible
	Traffic Demand[b]		
1	Demand between O-I and D-1	1 through 4	4I
2	Demand between O-I and D-2	5 through 8	4I
3	Demand between O-I and D-3	9 through 12	
		* * * * * * * * * * *	
16	Demand between O-I and D-16	61 through 64	4I

[a] One card for each subsection.

[b] Demand should be in 15-min rate of flow.

After reading input data for freeway subsection parameters and on-ramp volume limits, the program follows the travel time calculation procedure from the first time interval to the last. The flow diagram of the freeway travel time submodel is shown in Figure 2.

An example of output from the freeway travel time submodel is shown in Figure 3. The major portions of the output are (a) the input data of subsection parameters; (b) the summary of computing results; and (c) the travel time and the single trip travel time of each O-D movement.

The summary includes the section number; input demand; traffic volume, capacity, and weaving effect all in hourly rate of flow; v/c ratio; density in vehicles per mile; average speed in miles per hour; an asterisk for queuing identification, if any; subsection length and queue length both in feet; and excess demand in hourly rate of flow.

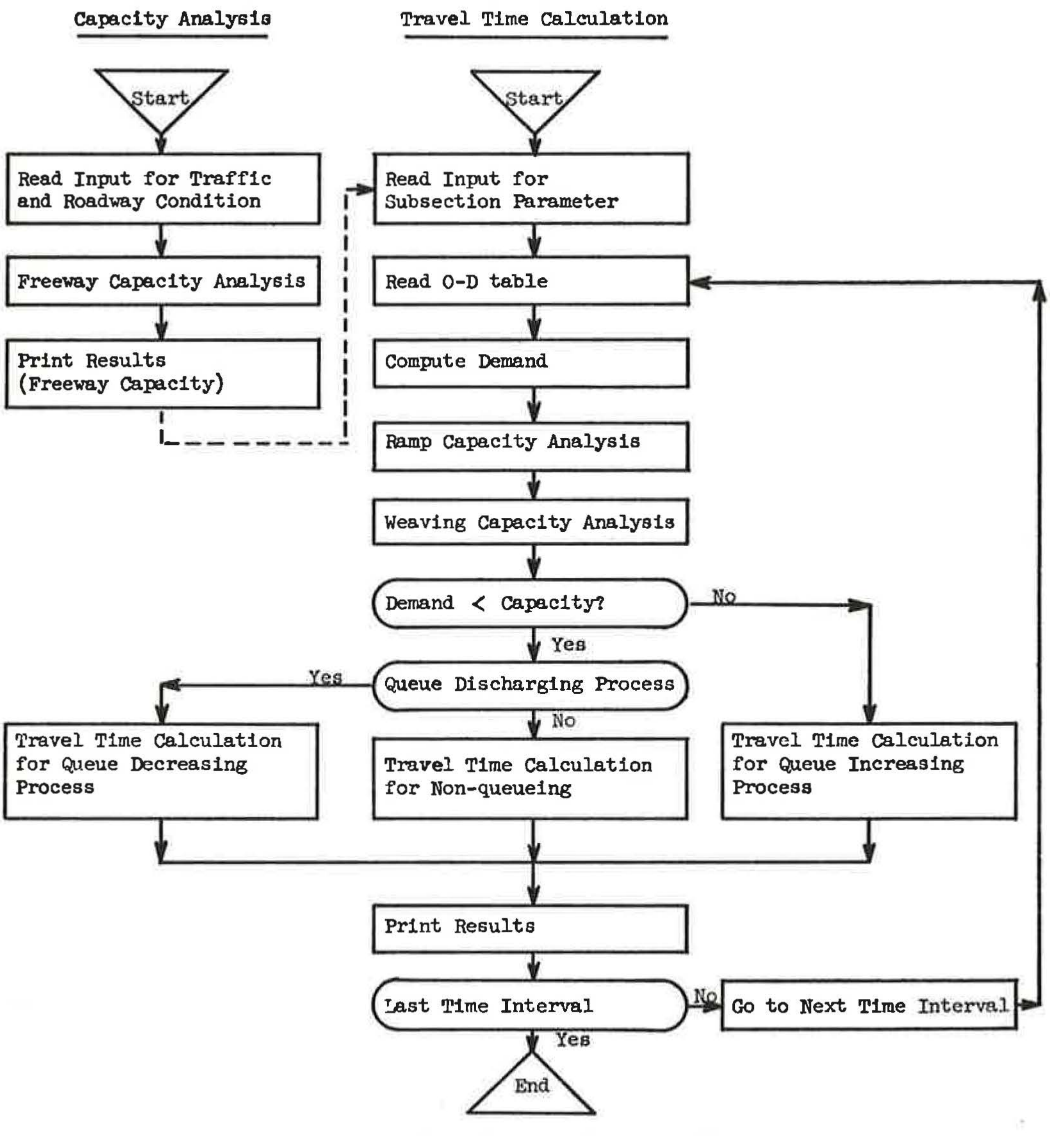

Figure 2. Flow diagram of freeway model.

INPUT DATA
30 SUBSECTIONS FAC(1)= 4.0 FAC(2)= 2.0

SSEC NO.	NO. LANE	CAP.	LENG	TRK. FAC.	ORG. DES.	LFT RMP	SUBSECTION LOCATION		
1	5	9434	1630	.943	OD	0	MAINLINE ORIGIN	TO	POWELL OFF
2	5	9453	1960	.961		0	POWELL OFF	TO	POWELL ON
3	5	9615	1550	.961	OD	0	POWELL ON	TO	ASHBY OFF
4	4	7619	1960	.952		0	ASHBY OFF	TO	ASHBY ON
5	4	7550	500	.952	O	0	ASHBY ON	TO	500 FEET PT.
6	4	7550	4790	.952	D	0	500 FEET PT.	TO	UNIVERSITY OFF
7	4	7692	3030	.961		0	UNIVERSITY OFF	TO	UNIVERSITY ON
8	4	7692	2160	.961	OD	0	UNIVERSITY ON	TO	GILMAN OFF
9	4	7563	2030	.961		0	GILMAN OFF	TO	GILMAN ON
10	5	9130	1250	.970	OD	0	GILMAN ON	TO	BUCHANAN OFF
11	4	7619	900	.952	D	1	BUCHANAN OFF	TO	HOFFMAN OFF(LEFT)
12	3	5671	1320	.961	D	0	HOFFMAN OFF	TO	PIERCE OFF
13	3	5750	720	.961		0	PIERCE OFF	TO	PIERCE ON
14	3	5806	2610	.970	OD	0	PIERCE ON	TO	CENTRAL OFF
15	3	5728	1660	.970		0	CENTRAL OFF	TO	CENTRAL ON
16	3	5806	1890	.970	OD	0	CENTRAL ON	TO	CARLSON OFF
17	3	5520	2310	.940		0	CARLSON OFF	TO	CARLSON ON
18	3	5950	1460	.980	OD	0	CARLSON ON	TO	POTRERO OFF
19	3	5806	3800	.970		0	POTRERO OFF	TO	CUTTING ON
20	3	5880	1100	.980	O	0	CUTTING ON	TO	GRADE CHANGE PT.
21	3	5950	660	.980	D	0	GRADE CHANGE PT.	TO	MACDONALD OFF
22	3	5950	1480	.980	D	0	MACDONALD OFF	TO	SAN PABLO OFF
23	3	5728	1480	.970		0	SAN PABLO OFF	TO	SAN PABLO ON
24	4	6850	800	.980	OD	0	SAN PABLO ON	TO	SOLANO OFF
25	3	5800	4690	.970	D	0	SOLANO OFF	TO	SAN PABLO DAM OFF
26	3	5806	2190	.970		0	DAM ROAD OFF	TO	DAM ROAD ON
27	3	5800	2320	.970	OD	0	DAM ROAD ON	TO	ROAD 20 OFF
28	3	5049	830	.850		0	ROAD 20 OFF	TO	GRADE CHANGE PT.
29	3	4746	1180	.793		0	GRADE CHANGE PT.	TO	ROAD 20 ON
30	3	4700	2560	.780	OD	0	ROAD 20 ON	TO	MAINLINE DESTINATION

RAMP LIMITS = 1500
OFF-RAMP 6 LIMIT= 2000

SUB SEC	..INPUT DEMAND... ORG	DES	TOTAL	VOL	FWRY CAP	WEAVE EFF	V/C	DENS	MPH	TRAV TIME	LENG	QUEUE FEET	NP
1	7648	1080	7648	7648	9434	0	.81	27	56	.32	1630	0	0
2	0	0	6568	6371	9453	0	.67	28	44*	.49	1960	1034	197
3	280	1040	6848	6747	9083	532	.74	74	18*	.96	1550	1549	197
4	0	0	5808	5692	7619	0	.74	79	18*	1.19	1960	1959	197
5	680	0	6488	6372	7550	0	.84	44	36*	.15	500	499	940
6	0	600	6488	6372	7550	0	.84	59	27*	1.97	4790	4789	940
7	0	0	5888	5688	7692	0	.73	83	17*	2.02	3030	3029	940
8	840	200	6728	6564	7571	121	.86	78	21*	1.11	2160	2159	940
9	0	0	6528	6336	7563	0	.83	79	20*	1.11	2030	2029	940
10	1080	760	7608	7416	7416	1714	1.00	42	35	.40	1250	0	0
11	0	1800	6848	6655	7442	177	.89	30	55	.18	900	0	0
12	0	80	5048	4899	5671	0	.86	29	55	.26	1320	0	0
13	0	0	4968	4824	5750	0	.83	29	55	.14	720	0	0
14	320	520	5288	5144	5806	0	.88	31	55	.53	2610	0	0
15	0	0	4768	4637	5728	0	.80	27	56	.33	1660	0	0
16	320	240	5088	4413	5806	0	.76	30	49*	.43	1890	260	544
17	0	0	4848	4186	5520	0	.75	45	31*	.82	2310	2309	544
18	240	400	5088	4426	5950	0	.74	73	20*	.79	1460	1459	544
19	0	0	4688	4041	5806	0	.69	89	15*	2.82	3800	3799	544
20	1000	0	5688	5227	5227	653	1.00	49	35	.35	1100	0	0
21	0	320	5688	5227	5297	653	.98	39	44	.16	660	0	0
22	0	480	5368	4937	5904	46	.83	29	55	.30	1480	0	0
23	0	0	4888	4502	5728	0	.78	26	56	.29	1480	0	0
24	800	440	5688	5302	6052	798	.87	24	55	.16	800	0	0
25	0	1280	5248	4893	5800	0	.84	29	55	.95	4690	0	0
26	0	0	3968	3705	5806	0	.63	21	57	.43	2190	0	0
27	200	600	4168	3905	5800	0	.67	22	57	.45	2320	0	0
28	0	0	3568	3345	5049	0	.66	19	57	.16	830	0	0
29	0	0	3568	3345	4746	0	.70	19	57	.23	1180	0	0
30	120	3688	3688	3465	4700	0	.73	20	56	.51	2560	0	0

			QUEUE LENGTH VEHICLES	DELAY 0.1 VEH-HRS
ON-RAMP	2	INPUT POINT	0	0
		MERGING POINT	0	30
		TOTAL	0	30
ON-RAMP	4	INPUT POINT	0	0
		MERGING POINT	0	11
		TOTAL	0	11
ON-RAMP	9	INPUT POINT	0	0
		MERGING POINT	51	184
		TOTAL	51	184

TRAVEL TIME FOR ONE TRIP .01 MINUTE

0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1	32	177	508	821	972	990	1016	1083	1159	1320	1653	1683	1728	1873	1911	2001
2	0	96	427	740	891	909	935	1002	1078	1239	1572	1602	1647	1742	1830	1920
3	0	0	212	525	676	694	720	787	863	1024	1357	1387	1432	1527	1615	1705
4	0	0	0	111	262	280	306	373	449	610	943	973	1018	1113	1201	1291
5	0	0	0	0	40	58	84	151	227	388	721	751	796	891	979	1069
6	0	0	0	0	0	0	0	53	129	290	623	653	698	793	881	971
7	0	0	0	0	0	0	0	0	43	204	537	567	612	707	795	885
8	0	0	0	0	0	0	0	0	0	79	412	442	487	582	670	760
9	0	0	0	0	0	0	0	0	0	0	51	81	126	221	309	399
10	0	0	0	0	0	0	0	0	0	0	0	0	16	111	199	289
11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	45	135
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	51

TOTAL TRAVEL TIME IN .01 VEH-HRS

0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1	144	767	1244	684	2802	4966	2201	191	637	506	1680	2244	1152	3585	1560	7603
2	0	0	21	0	44	166	0	150	0	61	0	0	82	232	122	832
3	0	0	0	0	0	497	36	249	129	221	67	0	119	432	215	1420
4	0	0	0	0	17	196	20	24	89	122	94	227	135	445	200	1506
5	0	0	0	0	6	51	0	80	22	206	72	187	106	356	179	1300
6	0	0	0	0	0	0	0	0	0	29	20	32	58	171	102	712
7	0	0	0	0	0	0	0	0	0	17	0	37	51	176	92	649
8	0	0	0	0	0	0	0	0	0	7	13	29	32	87	55	380
9	0	0	0	0	0	0	0	0	0	0	0	0	37	187	123	1044
10	0	0	0	0	0	0	0	0	0	0	0	0	3	75	63	606
11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	99
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	25

	CURRENT TIME INTERVAL VEH-HRS	PASS-HRS	CUMULATIVE VALUES VEH-HRS	PASS-HRS
FREEWAY TRAVEL TIME=	445.	614.	1386.	1912.
INPUT DELAY=	22.	31.	46.	63.
TOTAL TRAVEL TIME=	468.	646.	1432.	1976.

Figure 3. Output from the freeway travel time submodel.

The total travel time and the single trip travel time for each O-D movement are printed in the same format as that for the input O-D table. The total travel time is expressed in hundredths of an hour, and the travel time for one trip is expressed in hundredths of a minute. The total travel time expended in the whole freeway section during each time interval is printed under the 2 tables.

MODEL VALIDATION AND CONTROL PLAN EVALUATION

Model Section

The northbound evening peak traffic on the East Shore Freeway, which extends from the Eastbay Distribution Structure to Hilltop Drive interchange, was selected to develop and validate the freeway model. Comprehensive traffic data were collected from the freeway section through the Bay Area Freeway Operations Study, which has been conducted by the Institute of Transportation and Traffic Engineering. These data include volume counts on all on- and off-ramps, vehicle performance studies, and aerial photography. A complete inventory of the data utilized is presented in another report (5).

The freeway subsection parameters for the study section are shown in Figure 1. O-D information for the study section was collected through a postcard survey and a license-plate study.

Validation

System Validation—Validation of the computer program was undertaken to ensure that the model would derive similar results as compared to those obtained through manual calculation. For this purpose a freeway section was selected from the beginning of the study section to the Hoffman off-ramp. Manual calculations from this freeway section were made for both queue-increasing-time-interval and queue-decreasing-time-interval; the results are given in Table 3. The differences between the computed and the manually derived results would be caused either by round-off processes in the calculations or reading errors on graphs for v/c ratio and average speed relationship from the manual speed estimation method.

Validation Against Actual Traffic Data—Three variables were selected from the computer output and compared with the data collected from the study section: single trip travel time, total travel time, and traffic densities.

Figure 4 shows single trip travel time variation through the study section both from the model outputs and the actual data. At any time interval, the model outputs show much greater values than the actual conditions. For example, comparing the time interval between 5:00 p.m. and 5:15 p.m., the single trip travel time from the model output is about 35 percent greater than the actual value. In fact, the total travel time between 4:00 p.m. and 6:00 p.m. from the model output is 31 percent greater than the actual data. Figure 5 shows the total travel time expended within each 15-minute time interval as calculated from both actual data and model outputs.

TABLE 3

RESULTS OF SYSTEM VALIDATION

Time Interval and Traffic Situation	Calculation for	Manual Calculations	Computer Results	Percent Error
4th time interval, queue increasing	Queue length	4,404 and 14,102 ft	4,395 and 14,203 ft	0.02 0.7
	On-ramp delay	1.15 vehicle-hr	1.1 vehicle-hr	—
	Total travel time	228.3 vehicle-hr	228.0 vehicle-hr	—
7th time interval, queue decreasing	Queue length	5,310 ft	5,354 ft	0.8
	On-ramp delay	0.0 vehicle-hr	0.0 vehicle-hr	—
	Total travel time	196.8 vehicle-hr	197.0 vehicle-hr	—

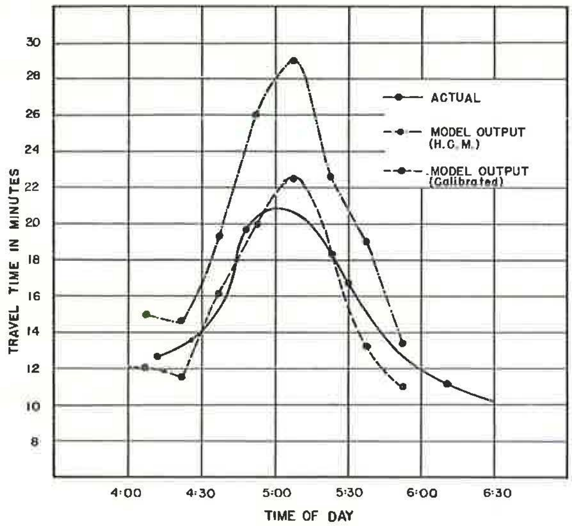

Figure 4. Single travel time versus time of day.

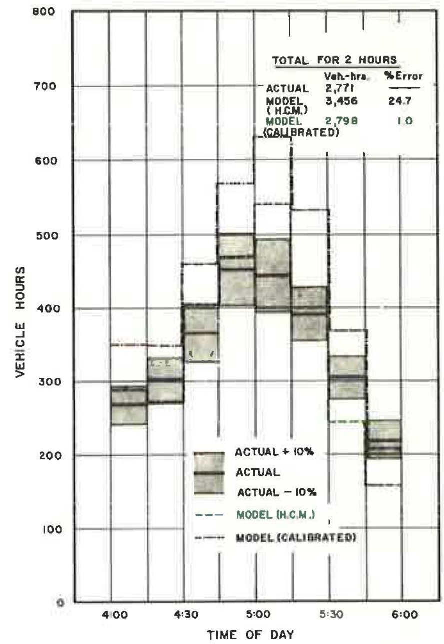

Figure 5. Total travel time versus time of day.

Figure 6 shows density contour maps based on both actual data and model outputs. Comparing the actual contour map developed from aerial photographs, it was found that the model could locate 3 actual major bottlenecks on the East Shore Freeway: (a) subsection 5 in the vicinity of the Ashby interchange, (b) the weaving section between the Gilman on-ramp and the Buchanan off-ramp, and (c) the weaving sections between the Cutting on-ramp and the MacDonald off-ramp. Even though the model gave longer congestion time periods for those major bottlenecks, it could not indicate minor bottlenecks such as the weaving section between the Carlson on-ramp and the Potrero off-ramp.

Refinement and Reevaluation

Refinement—Comparing the model output with actual data, it is apparent that the model cannot be used without some modification. Considering the fact that the model always estimates greater travel times and longer congestion time periods, it was assumed that during evening peak periods drivers might travel on the study section with higher speeds than the speed estimated from Figure 9.1 of the Highway Capacity Manual, and that the actual capacities of the study section would be little different from the capacities calculated strictly following the Highway Capacity Manual. Consequently, the steps taken to refine the model were (a) adjustment of the relationship between v/c ratio and speed, and (b) capacity adjustment.

Relationship Between Speed and v/c Ratio—For the nonqueuing situation, the average speeds calculated from the actual data are plotted for the v/c ratio of corresponding subsections and time intervals. Then, the relationship between speed and v/c ratio was fitted by a straight line and parabola as shown in Figure 7. On the other hand, because there were not enough data for the queuing situation, no adjustments were made for the lower part of the curves, which represents the relationship between v/c ratio and the speed for the queuing situation.

Capacity Adjustments—Capacities at bottleneck sections were estimated from both the actual freeway volume counts and the relationship between bottleneck subsection demands and the length of congestion periods. Then, with these adjusted capacities, the model was tested again. Comparing the test-run results with actual data, capacity adjustments were made at certain sections where the model produced physical

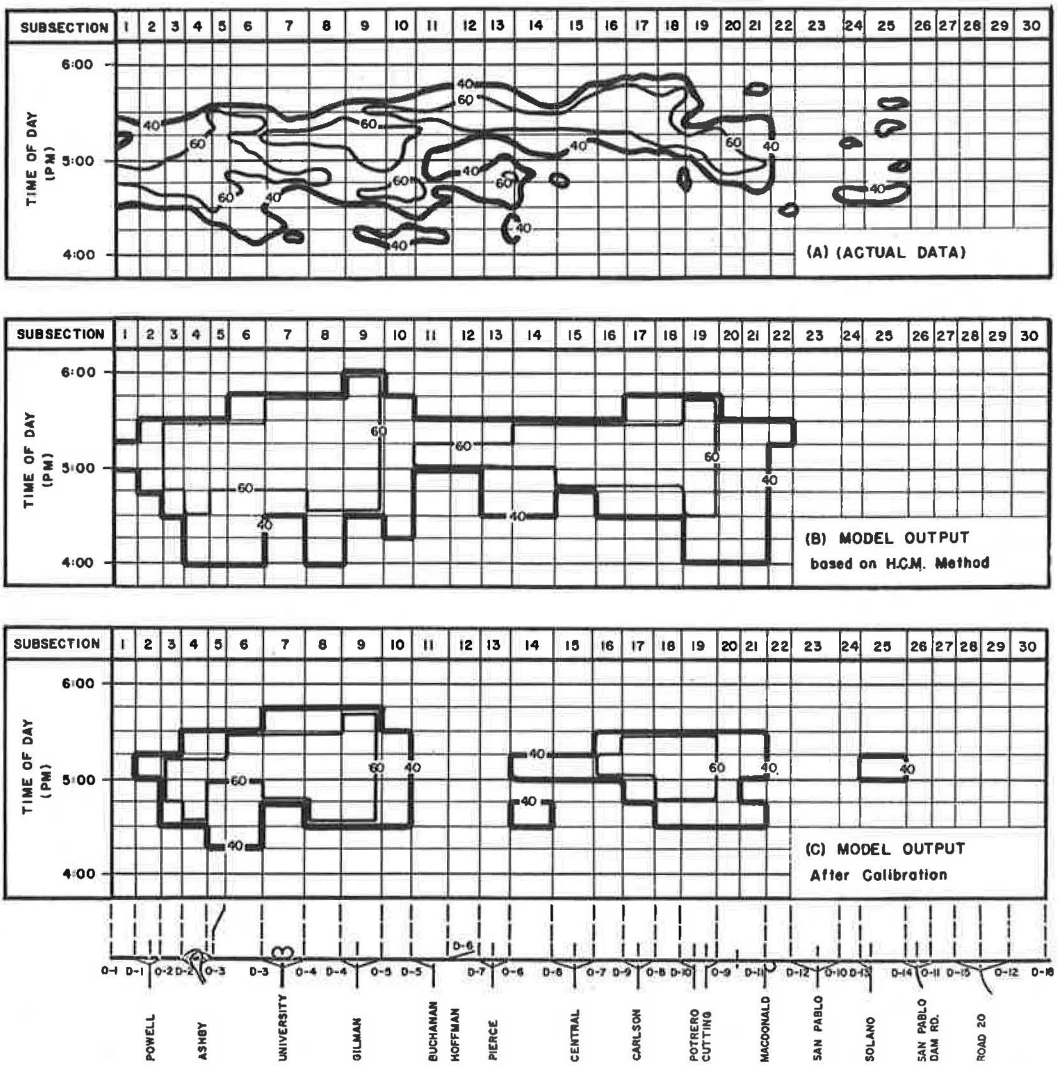

Figure 6. Density contour maps.

queues, which should not occur judging from the actual data. These new capacities are only slightly different from the original capacities calculated through the Highway Capacity Manual method; the differences are between -1 and +4 percent.

Reevaluation—The model was tested with the adjusted capacities and the new relationship between v/c ratio and the average speed. The results are shown in Figures 4, 5, and 6.

Figure 4 shows that for any given time interval the single trip travel time with adjusted capacities and the new speed relationship is within a ±15 percent error.

Figure 5 shows the total travel time variation with time for both the model output and the actual data. The computer outputs for total travel time are within a ±10 percent error for 5 out of 8 time intervals. Generally the results show greater values of total travel time for before-peak time intervals and smaller values for after-peak time intervals. This deviation is caused from the instantaneous demand propagation, which is one of the basic assumptions of this model described earlier. Anyhow, the total travel time between 4:00 p.m. and 6:00 p.m. was estimated with less than one percent error.

Figure 6 shows the density contour map. Minor bottlenecks could not be located even with adjusted capacities. However, for 3 major bottlenecks, the model shows reasonable congestion time periods.

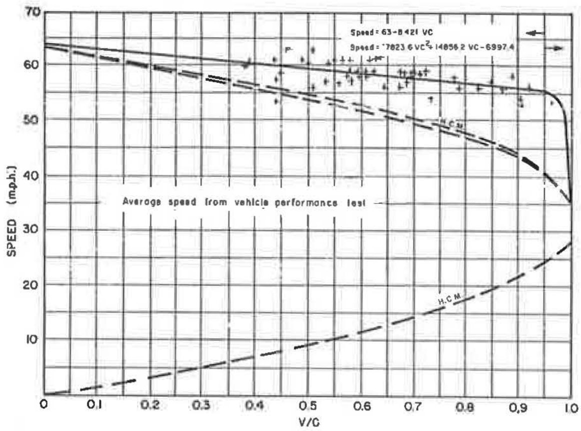

Figure 7. Relationship between v/c ratio and average speed.

Application

The freeway model was finally applied to evaluate the freeway improvement plans along the East Shore Freeway. Two improvement plans were tested to show how the model reacts for different traffic and roadway conditions. These two plans are (a) adding an auxiliary lane between the Cutting on-ramp and the MacDonald off-ramp, and (b) having a ramp closed at the Gilman on-ramp between 4:30 p.m. and 5:15 p.m.

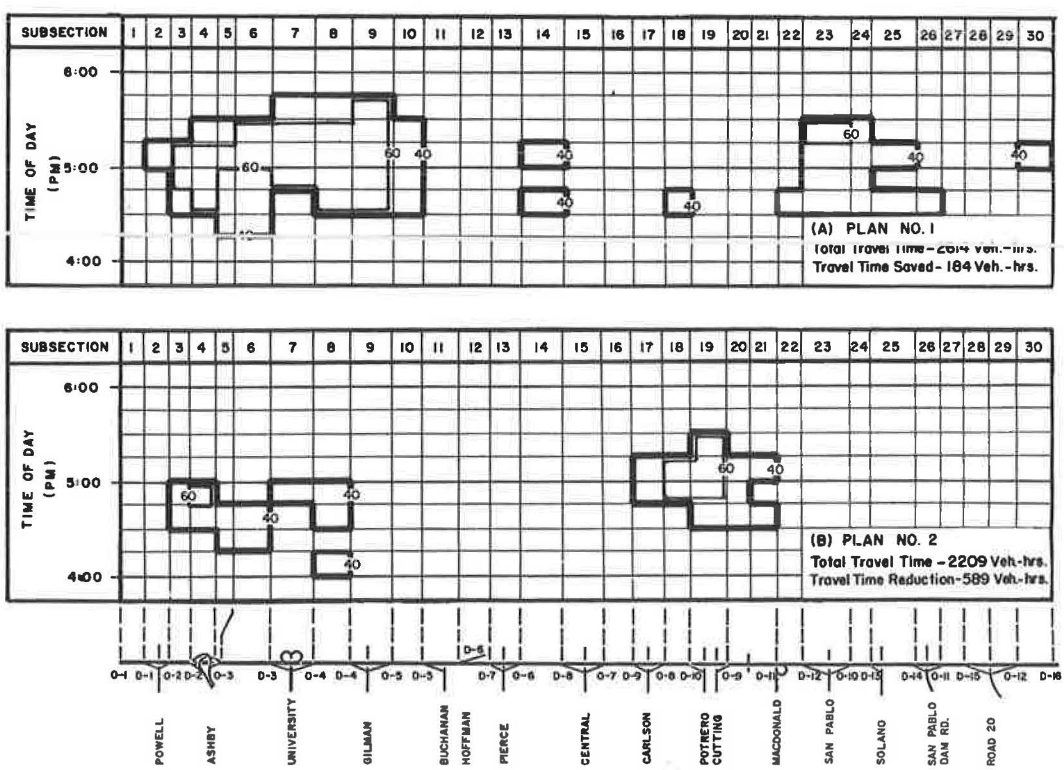

Figure 8. Test improvement plan evaluation.

Figure 8a shows the results for plan 1. The bottleneck between Cutting and MacDonald is almost eliminated, but other bottlenecks are produced downstream. Total travel time saved is 184 vehicle-hours.

Figure 8b shows the results of plan 2. The bottleneck between Gilman and Buchanan is almost eliminated. However, the downstream bottlenecks cannot be eliminated. Reduction in total travel time is about 589 vehicle-hours. In this case, travel time increase for diverted traffic is not considered. It is necessary to give it consideration on an associated surface street network.

CONCLUSIONS

The experience gained in working with development of the freeway model led to the following conclusions.

1. The freeway model based on the Highway Capacity Manual gives estimates of travel times for both nonqueuing and queuing situations. The model can be used to evaluate the effectiveness of freeway operation improvement plans.
2. The length of the freeway sections should not be too long. Judging from the outputs of single trip travel times, a reasonable section length is between 5 and 10 miles.
3. The following possible future research might further develop the systematic and analytical evaluation procedure: establish systematic and reasonable procedures for adjusting capacities, and develop a procedure to evaluate the movement of diverted traffic on related network systems.

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Measures of Effectiveness for Urban Traffic Control Systems

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This paper describes a portion of the analyses performed by TRW Systems under a contract for development of a second-generation surveillance methodology for use in a computerized system. The second-generation methodology, for use in the 1970's, provides more detailed and accurate information than the first-generation systems installed in Toronto and Wichita Falls. A primary function of a surveillance system is to evaluate the performance of the traffic system. Therefore, the first step in the development of a surveillance methodology is the definition of system objectives and related measures of effectiveness (MOE). This task is the subject of this paper. A review of literature concerned with MOE and computerized surveillance and control was performed; only the resulting list of MOE is presented in this paper. System objectives of maximization of the amount of service and maximization of the quality of service were identified, where the quality of service includes the smoothness of flow. A list of MOE evaluation criteria was compiled and used to systematically reduce a candidate set of approximately 40 MOE encountered in the literature to the following 3 recommended MOE: travel time, service rate (also called total travel) computed as the product of volume and link length, and ratio of effective to spot kinetic energies. Energy ratio was not encountered in the literature, but was developed during the study as a readily obtainable measure for flow smoothness. The traffic parameters required to compute these MOE are specified. Presentation of data and its use for control are also considered.

•THE URBAN TRAFFIC CONTROL SYSTEM (UTCS) of the 1970's will be burdened with increasing demands for more effective traffic control. In order to more accurately evaluate the effectiveness of improved control, a second-generation surveillance methodology must be developed. The methodology specifies the procedures for automatically collecting traffic data in an urban network, transmitting it to a central digital computer, and processing it for system evaluation purposes.

The dual functions of a surveillance system are to evaluate the performance of the traffic system and to supply accurate data for system control. An initial phase in developing the second-generation surveillance methodology is the selection of appropriate measures of effectiveness (MOE) with which to satisfy the dual surveillance functions. This selection is the subject of this paper.

The first step in the effort was a review of the following subjects: existing and planned computerized surveillance and control systems, surveillance and measurement techniques, and measures of effectiveness. The bibliography and results of the review may be found in other reports (1, 2) and are not duplicated in this paper.

The next step was the definition of the objectives of a traffic control system and establishing criteria with which to evaluate MOE. By testing the various measures of

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effectiveness with respect to the objectives and criteria, the large set of candidate MOE were reduced to the recommended set of a few MOE. The traffic parameter measurements required to compute the recommended measures then were determined. The equally important factors of presentation of data and its use for control purposes also were considered. Another report (2) provides additional details not found in this paper.

TRAFFIC OBJECTIVES AND CANDIDATE MOE

The general objective of an urban traffic control system is to utilize the existing street system most effectively. The primary function of a city's streets is to provide for the safe and efficient movement of persons and goods. These statements are qualitative and must be related to quantitative criteria that are capable of being measured and optimized in a real-time computer-controlled traffic system. For example, "safe movement" implies minimum accidents where accidents, in many urban cases, are related to the jerkiness of flow (quality of service). Similarly, "efficient movement" can imply maximum flow or speed, minimum delay or fuel consumption, or some nonredundant combination (amount of service and cost). From this discussion, 3 objectives can be defined that represent differing viewpoints on what constitutes effective street system utilization. They are maximization of service, optimization of quality of service, and minimization of cost. The first 2 objectives are the more basic. MOE associated with them can be converted into an economic measure by applying appropriate cost factors.

Objective 1 corresponds to maximizing the traffic movement (i. e., amount of traffic moved or speed). Objective 2 is somewhat more complex in that both the traffic movement and the smoothness of the flow must be considered. Very few of the MOE encountered in the literature attacked this objective directly. Objectives 1 and 2 can be expressed in terms of the following 2 functional objectives: maximization of traffic movement and maximization of flow smoothness.

It was convenient to classify the MOE encountered in the literature according to functional objective with categories under each. Table 1 gives the list of candidate MOE. The empirical indexes of Greenshields and Platt appear twice because they attempt to combine both functional objectives (i. e., optimize the "quality" of service). An MOE not found in the literature, the energy efficiency (ratio of effective to free-flow kinetic energy), is included in the flow smoothness group.

MOE EVALUATION CRITERIA AND EVALUATION

In order to systematically reduce the set of candidate MOE to the much smaller set of recommended MOE, evaluation criteria must be established and applied to the candidates. Figure 1 shows a conceptual representation of the approach for accomplishing the reduction. For the purposes of this study, typical evaluation criteria are relevance to system under consideration, ability to quantify relationships, practicability of measurements and/or computations, ease of establishing a reference optimum, sensitivity and validity of indications, and redundancy and/or equivalence. Some MOE can be eliminated on the basis of a single evaluation criterion, while others are eliminated through a combination of criteria.

Relevance to System Under Consideration

There are 3 general groupings of MOE for applicability to system evaluation. The groupings refer to MOE that are applicable to system element, but not to total system; both element and system; and total system, but not to element. The middle group contains the majority of MOE.

For a general urban network, several MOE are too specialized to indicate effectiveness under varying conditions. These can be eliminated as system MOE, although a few may be useful on the microscopic level. These MOE include main or side street delay, mean speed on slowest link, minimum individual speed on slowest link, maximum individual delay in queue per intersection, mean queue per intersection, maximum queue length per intersection, and maximum individual travel time.

TABLE 1

CANDIDATE MEASURES OF EFFECTIVENESS

Functional Objective	Category	Characteristic
Traffic movement	Delay	Total in system
		Mean in system
		Aggregate individual in system
		Aggregate individual in queue
		Main street
		Side street
		Mean in worst link
		Mean in queue in worst link
		Maximum in queue in worst link
		Proportion delayed at worst intersection
		Delay rate
	Stopped/queued	Total queue in system
		Mean queue in system
		Mean queue at worst intersection
		Maximum queue at worst intersection
		Proportion stopped
	Travel time	Total in system
		Mean in system
		Mean individual through system
		Maximum individual through system
		Mean on slowest link
		Maximum on slowest link
	Speed	Overall mean in system
		Individual mean through system
		Individual minimum through system
		Mean on slowest link
		Minimum on slowest link
		Spot speed
	Volume (flow)	Vehicles/unit time
	System occupancy	Sensor on-time/total time
	Rothrock and Keefer's congestion index	Actual occupancy/optimum occupancy
	Density	Vehicles/unit length
	Service rate	Total travel, vehicle-miles/hour
	Greenband width	
	Cycle failure	
	Kinetic energy	
	Greenshields' index[a]	
	Platt's index[a]	
Flow smoothness	Acceleration noise	Standard deviation of acceleration $\sigma_a = \left[\frac{1}{T}\int_0^T a^2 dt\right]^{1/2}$
	Mean velocity gradient	σ_a/mean speed
	Energy efficiency	Ratio of effective kinetic energy to measured (free-flow) kinetic energy
	Greenshields' index[a]	
	Platt's index[a]	

[a]Included in both objectives because they attempt to combine both objectives.

The Set of Candidate
Measures of Effectiveness

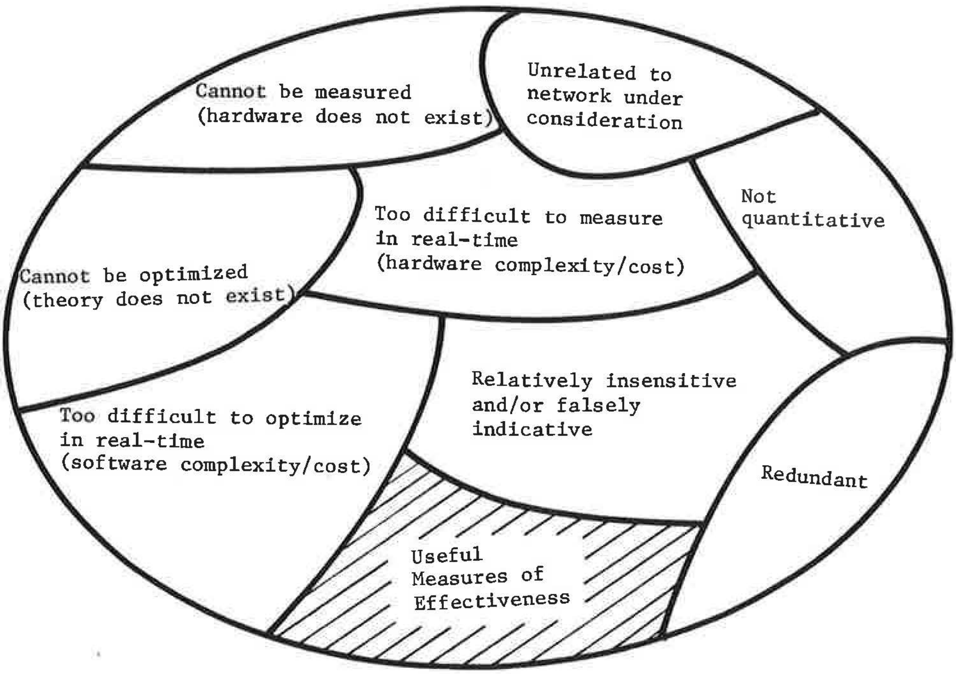

Figure 1. Feasibility classification of measures of effectiveness.

Ability to Quantify Relationships

Certain qualitative measures (e. g., congestion) elude numerical representation, but all of the MOE listed can be expressed quantitatively. However, accuracies are related to difficulty of measurement, computation, and/or estimation.

Practicability of Measurements and/or Computations

The least desirable MOE are those that cannot be measured or accurately estimated automatically. A major restriction is that data collected via roadway instrumentation are applicable only on a block-by-block basis; it is not feasible to track uninstrumented vehicles through the complete system. On this basis, the following MOE may be eliminated: aggregate individual delay in system, aggregate individual delay in queue, maximum individual travel time, and minimum individual speed in system. This restriction also requires that all mean values (e. g., mean travel time) be defined as the mean from data per link, rather than the mean of individual vehicles traveling through the complete system.

Greenshields and Platt's indexes require measurement of quantities (steering wheel reversals, brake applications, and vehicle direction changes) that are not feasible to obtain continuously. However, the occasional use of an instrumented test car to check and/or calibrate a roadway surveillance system appears desirable.

Measurement of acceleration noise and mean velocity gradient is feasible but relatively difficult and so is classified as undesirable.

Ease of Establishing a Reference Optimum

It is desirable that an MOE possess an easily defined reference optimum that is independent of traffic, geometric, and climatic conditions. An MOE that is always to be minimized or maximized is more desirable than an MOE whose optimum value changes with operating conditions. In addition, MOE with an optimum value probably are not basic because their optimum must be defined in terms of other parameters. MOE classified undesirable on the basis of these considerations include Rothrock and Keefers' congestion index, greenband width, density, kinetic energy (but not energy ratio), and possible system occupancy. MOE having an easily defined reference value (e.g., zero or one) and that are to be minimized or maximized are desirable.

Sensitivity and Validity of Indications

The relationship between an MOE and traffic conditions should be essentially unique; i.e., widely differing traffic conditions should not result in the same MOE value or trend. On this basis, the use of volume (flow) alone as an MOE was eliminated; however, the use of volume in conjunction with another parameter (e.g., speed) may be meaningful.

The use of either proportion delayed or proportion stopped alone can also be misleading. Studies (3) showed that a single-dial fixed-time controller produced the smallest proportion stopped, but also the largest system delay.

This sensitivity/validity criterion also points out the need to consider the worst-case microscopic conditions in addition to system-wide totals or averages. For example, if mean travel time is used, the travel time on the worst link must be considered to eliminate unduly large delays.

To ensure the validity of indications, it appears necessary that the surveillance system extend beyond the major control area. In this way, the surveillance system would take into account the queues that may build up while attempting to enter or leave the controlled area.

Redundancy and/or Equivalence

At this stage of MOE evaluation, the candidates remaining for determining system effectiveness are delay (total, mean), delay rate, queue length (total, mean), travel time (total, mean), mean speed in system, service rate, volume, cycle failure, acceleration noise, mean velocity gradient, and energy ratio where, as indicated previously, certain of these MOE cannot be used alone. Other MOE evaluation criteria such as ease of interpretation and ease of conversion to economic terms may be listed; but these are, in some sense, implicit in the previous evaluation criteria.

The next step is to seek relationships between MOE and eliminate the redundant measures. The choice of a particular MOE from a set of correlated MOE involves some subjectivity. The choices presented here have been based largely on 2 requirements: (a) The MOE chosen should be amenable to use in an on-line optimization procedure; and (b) the related MOE should be obtainable from the chosen MOE using no additional data and little additional computation.

Those MOE concerned with the objective of maximizing traffic movement are considered first. Table 2 (3) gives some correlations between MOE as obtained by simulation of traffic at a single intersection.

Figure 2 shows some general interrelationships. Cycle failure can be determined, assuming consistent arrival rates, on the basis of the queue length at the start of green and the length of green; consequently, it can be eliminated as redundant with queue length. Now consider travel time, but recall that measurements can be made only on a link-by-link basis. Travel time is defined as the difference in the time a vehicle exits a link and the time it enters. If an ideal (free-flow or free-speed) travel time is defined, the difference between effective and ideal travel times is delay. Thus, minimizing travel time also minimizes delay (and delay rate). To accurately estimate the effective travel times of vehicles through a link, queue length information is necessary. Therefore, queue length and delay are implicit to travel time.

Now consider speed. The speed measured at a point is not necessarily representative. A more meaningful quantity is the effective speed through a link and is defined by

$$S = \frac{K}{T}$$

TABLE 2

CORRELATIONS BETWEEN MOE

MOE_1	MOE_2	Correlation Coefficient	Standard Deviation[a]	Comments
Mean system delay	Mean stopped delay	0.998	0.014	Linear; independent of signal control[b]
Mean system delay	Mean queue length	0.996	0.023	Linear; independent of signal control
Mean system delay	Mean delay in queue	0.998	0.018	Linear; independent of signal control
Mean system delay	Proportion of vehicles stopped	0.960	Not computed	Nonlinear; a function of signal control
Maximum individual delay	Mean system delay	0.970	0.540	A function of signal control
Maximum individual delay	Maximum stopped delay	0.997	0.170	Linear; independent of signal control
Maximum individual delay	Maximum queue at intersection	0.959	0.620	A function of signal control

Note: Based on microscopic simulation of an individual isolated intersection (3).

[a]Square root of mean square error between data points and curve fit. Larger values indicate large random variations or improper order of curve fit.

[b]"Independence" of signal control as used here is based on a limited examination of fixed time versus queue-dependent control schemes.

where T is the effective travel time defined in the preceding paragraph and K is a conversion constant. Thus, minimizing travel time maximizes speed and the two are redundant.

Based on the interrelationships just discussed, travel time is selected as a basic MOE; but it alone does not give the complete picture. The objective of maximum traffic movement involves 2 factors: the rapidity of movement, and the amount of traffic being moved (or the degree of utilization of the street system). Travel time provides a measure of the rapidity of movement. A measure of the amount of traffic processed is also

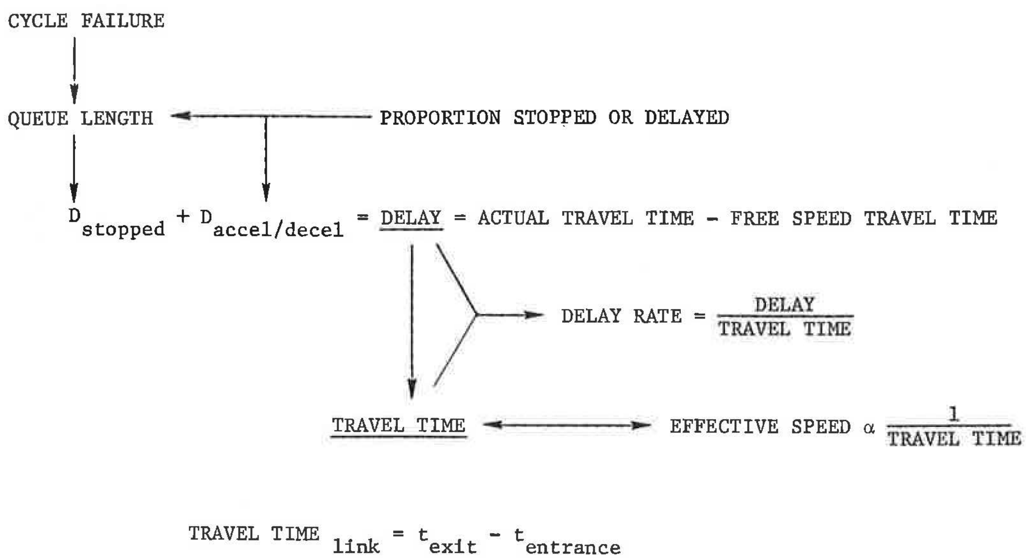

Figure 2. Interrelationships between traffic movement MOE.

necessary. The commonly used measure is volume; however, volume does not account for the distance traveled by vehicles. A more desirable MOE is the service rate (also called total travel) defined in the following.

Consider a system conceptually as a single source and a single sink separated by miles of roadway. The population of the source will be emptied into the sink most rapidly if the service rate (total travel) defined by

$$R = \text{vehicle-miles/hour}$$

is maximized. For a general urban network with many sources and sinks, it can be approximated on a link basis using volume and link length; i. e.,

$$R_{link} = (\text{volume})\ (\text{link length})$$

A common flow smoothness MOE is acceleration noise; however, it is difficult to measure with roadway instrumentation. It can be shown, based on a fluid-flow analogy, that acceleration noise is a measure of lost energy in the system. Another measure of lost energy (rather, energy efficiency) is the ratio of effective-to-measured kinetic energies. It has the advantage that it can be obtained from the measurements used to compute travel time.

Consider 2 kinetic energies given by

$$E_{eff} = \rho S_{eff}^2, \quad E_{meas} = \rho S_{meas}^2$$

where ρ is the density (vehicles/unit length), S_{eff} is the effective speed computed by S_{eff} = distance/travel time, and S_{meas} is the free-flow spot speed as measured by sensors. The difference between these 2 energies corresponds to an energy loss due to acceleration, deceleration, and waiting. Because the density is the same in both cases, the energy loss can be minimized by maximizing the energy ratio

$$\eta_E = \frac{E_{eff}}{E_{meas}} = \left(\frac{S_{eff}}{S_{meas}}\right)^2$$

If traffic is flowing smoothly with no stops, the efficiency becomes

$$\eta_E \approx 1.0$$

If the flow is interrupted by deceleration and stops, S_{eff} and η_E decrease.

A correlation between acceleration noise and energy ratio would be expected because both provide a measure of lost energy; Figure 3, presenting data from an arterial simulation, shows this correlation. Energy ratio was selected over acceleration noise (and the related quantity, mean velocity gradient) because it is easier to obtain. Energy ratio also reflects the effects of the number and length of stops. Thus, energy ratio appears preferable for measuring flow smoothness.

THE RECOMMENDED MOE

The set of candidate MOE has been reduced to the following recommended quantities:

Traffic movement MOE	Travel time, T
Flow smoothness MOE	Energy ratio, η_E
System utilization parameter	Service rate, R

It is felt that these quantities provide a specific decomposition of the general system objectives into the basic functions of rapidity of movement, flow smoothness, and street utilization.

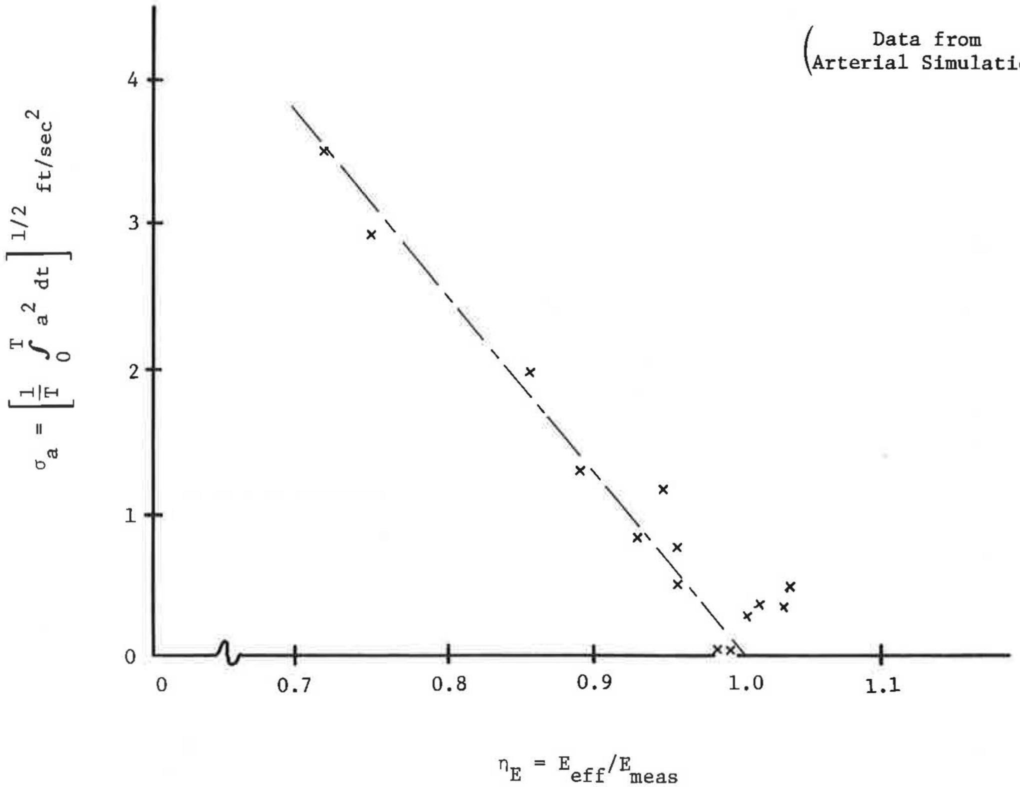

Figure 3. Relationship between flow smoothness MOE (acceleration noise versus effective-to-measured kinetic energy ratio).

When speaking of travel time, we refer to the actual (estimated) time for a vehicle to traverse a link (from past the upstream stop line until it passes the downstream stop line). The traffic movement MOE, travel time, can be thought of either as an average travel time per vehicle or as the total travel time (sum of travel time for all vehicles in system). The 2 versions provide complementary information. Total travel time will yield the change in performance of a system in terms of total hours so that cost factors may be easily applied to convert the performance change to monetary terms. Average travel time provides a measure of the benefit (or detriment) a typical individual will experience. Both total and average travel times are available from the same data.

Energy ratio (the ratio of effective-to-measured kinetic energies) is considered to be a meaningful flow smoothness MOE. It is computed using the square of the ratio of effective speed (reciprocal of travel time) to free-flow speed (measured). It is easier to measure than its correlated counterpart, namely, acceleration noise (requiring sophisticated instrumentation of individual cars).

There are 2 versions of service rate (total travel). Service rate computed as the product of output volume (count) and link length provides a meaningful measure of service that has been provided to vehicles. The product of input volume and link length would be a measure of service that must be provided to vehicles. The two will be equivalent unless there are major source/sinks within the link.

Some analyses and numerical results concerning the recommended MOE may be found in another report (2).

It is realized that some of the MOE may be unfamiliar to many traffic engineers, energy ratio in particular. However, it is emphasized that the recommended MOE inherently contain either the more common parameters or the information necessary to compute them. Consequently, if a traffic engineer desires other parameters (e.g., volume or number of stops), these can be displayed in addition to the recommended MOE.

TRAFFIC PARAMETER REQUIREMENTS

Computation and/or estimation of the recommended quantities require measurement of certain traffic parameters. The necessary parameters depend, to some extent, on the type of traffic flow being measured.

Naturally, current signal status and projected change times are necessary. In addition, it is desirable to know the arrival rates from major sources/sinks (flow at the system boundary and from/to major parking facilities). In addition to having geometrical information, other traffic parameters must be measured in order to compute/estimate the system evaluation quantities. The necessary parameters are functions of the traffic characteristics; i.e., laminar and turbulent flow require different instrumentation.

Idealized laminar flow has traffic free flowing at constant speed; there are no stops, queues, lane changing, or parking to interfere with the flow. Under these conditions, only speed and count need to be accurately measured. In turbulent flow (the opposite of laminar), the computation/estimation problem is more difficult. Free-flow speed and count are still necessary, but the accuracy requirements on speed are not as stringent. Instead, information is needed about the timing of events and the net result of turbulence (between the upstream speed/count instrumentation and the downstream stop line). To satisfy this need, queue (presence) data during the red phase and time-tagging of events are necessary for calibration/rectification purposes.

If travel time is accurately computed/estimated using this information, the effective travel time of a vehicle yields its energy ratio and contribution to service rate. Free-flow spot speed (used to estimate travel time) and effective speed (reciprocal of travel time) allow computation of the energy ratio. Knowledge of when a vehicle entered a link (time-tag on speed/count) plus effective travel time yields the estimated exit time; this exit time indicates when the vehicle has been "serviced" through the length of the link. Thus, travel time together with the measurements necessary to compute it yield the other system evaluation parameters.

The set of required traffic parameters are as follows:

1. Measurements—free-flow spot speed (and time at which vehicle crosses sensor), count (of vehicles entering), queue status (presence indications during red), signal state (current status and projected phase changes), and arrival rates (flow from major sources/sinks).
2. Computations/estimations—effective travel time (of vehicles through link, including effects of signal and vehicles ahead), energy ratio, and service rate (total travel based on vehicles serviced out of link).

COMPUTATION AND PRESENTATION OF SYSTEM EVALUATION DATA

The recommended MOE must be computed using data gathered from the urban network. Presented here are general definitions and formulas.

It is necessary to define a basic roadway segment or "link" for which data are available. A link is an instrumented portion of roadway, between 2 signalized intersections, on which traffic moves in only one direction. Typical link definitions are shown in Figure 4. Next, the urban traffic network "system" must be defined as a collection of interconnected links. Associated with this system of links are the quantities given in Table 3. The time-varying quantities are assumed to be available at distinct, equally spaced points in time. Using these quantities, the proposed formulas to compute MOE and system parameters over a time period of interest are given in Table 4. Only link and system quantities are shown; data for intersections and arterials are obtained by combining appropriate link information.

In addition to presenting information for the system as a whole, it will be necessary to examine the data for critical points within the system. In this way, critical inter-

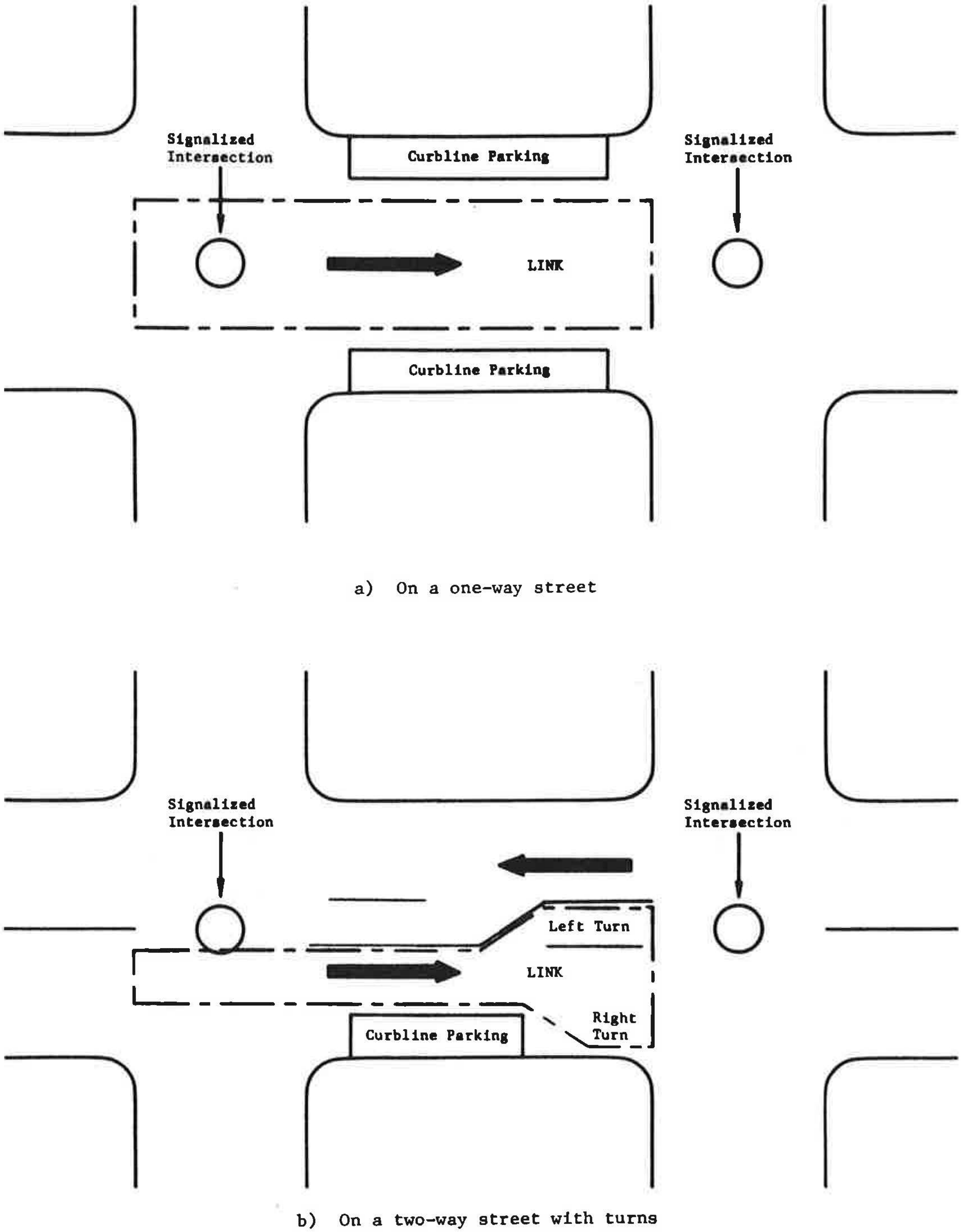

Figure 4. Definitions of simple links.

TABLE 3

NOTATION FOR FORMULAS

Symbol	Description
i	Subscript designating link in system
N	Total number of links in system
C_i	Number of vehicles within link i (including moving and queued but not parked cars) during period of interest[a]
$C_T = \sum_{i=1}^{N} C_i$	Total number of vehicles in system during period of interest
L_i	Lane-miles of roadway on link i
OC_i	Number of vehicles that exit link i (output count) during period of interest
j	Sub-subscript designating vehicle j on a link during period of interest
T_{i_j}	Effective travel time for vehicle j through link i including effects of signal and vehicles ahead (estimation converted to travel time over a standard/reference link)
$S_{i_j} = K/T_{i_j}$	Effective speed of vehicle j through link i (K is a units conversion factor)
$S^*_{i_j}$	Actual/measured free-flow speed of vehicle j in link i
	Link and System Summaries
TTT	Total effective travel time
$\overline{TT}$	Average effective travel time
R	Service rate (total travel based on output count)
$\overline{\eta}_E$	Average energy ratio (ratio of effective-to-measured kinetic energies)

[a]The "period of interest" may, for example, be a data-smoothing period of about one signal light cycle.

sections and major arterials are recognized as such and monitored accordingly. The MOE should be presented in real time, and also stored for later use in off-line analyses.

The MOE recommended must be presented in such a manner that system evaluation is facilitated. Generally, there are 2 ways of presenting system evaluation data, namely, parameters versus time, and one parameter versus another. The second type of plot makes time a hidden variable to illustrate the functional relationship between parameters; it has the advantage of presenting directly the "operating characteristics" of the system.

One interesting application of the second method is the cross-plotting of MOE for control evaluation. By plotting an MOE that indicates traffic quality versus a parameter that indicates system utilization, the operating characteristic of the system as a function

TABLE 4

MOE AND SYSTEM PARAMETER FORMULAS

Summaries	Formulas	Summaries	Formulas
Link	$TTT_i = \sum_{j=1}^{C_i} T_{i_j}$	System	$TTT = \sum_{i=1}^{N} TTT_i$
	$\overline{TT}_i = \frac{TTT_i}{C_i}$		$\overline{TT} = \frac{TTT}{C_T}$
	$\overline{\eta}_{E_i} = \frac{1}{C_i} \sum_{j=1}^{C_i} \left(\frac{S_{i_j}}{S^*_{i_j}} \right)^2$		$\overline{\eta}_E = \frac{1}{C_T} \sum_{i=1}^{N} C_i \times \overline{\eta}_{E_i}$
	$R_i = OC_i \times L_i$		$R = \sum_{i=1}^{N} R_i$

of utilization is obtained. (For example, travel time and energy ratio are functions of the effectiveness of control and can have various values for the same value of service rate.) If maximizing the quality-dependent MOE is our criterion, then examination of the operating characteristics allows selection of the better control method (or selection of regions where one control method is better than another). Figure 5 shows this operating characteristic concept.

The operating characteristic concept simplifies the problem of presenting the data. Because service rate is indicative of the amount of traffic being served in the system, a logical choice is to plot travel time and energy ratio as functions of service rate; that is, travel time ($\overline{TT}$ and/or TTT) versus service rate (R), and energy ratio ($\overline{\eta}_E$) versus service rate (R).

In order to present all 3 MOE on a single plot, a linear combination of travel time and energy ratio can be formed, for example,

$$J = \alpha \times \overline{TT} - \beta \times \overline{\eta}_E$$

where α and β are weighting factors at the disposal of the traffic engineer. The quantity J would be plotted as a function of service rate and, as with the individual MOE, monitored both on the system-wide and worst-element basis.

REAL-TIME CONTROL

The parameter J can also be used for on-line optimization and control because one purpose of real-time surveillance is to provide a "payoff function" (a quantity to be extremized). However, the requirements for the payoff function are slightly different for control purposes than for evaluation. In evaluation, it is necessary to have measures

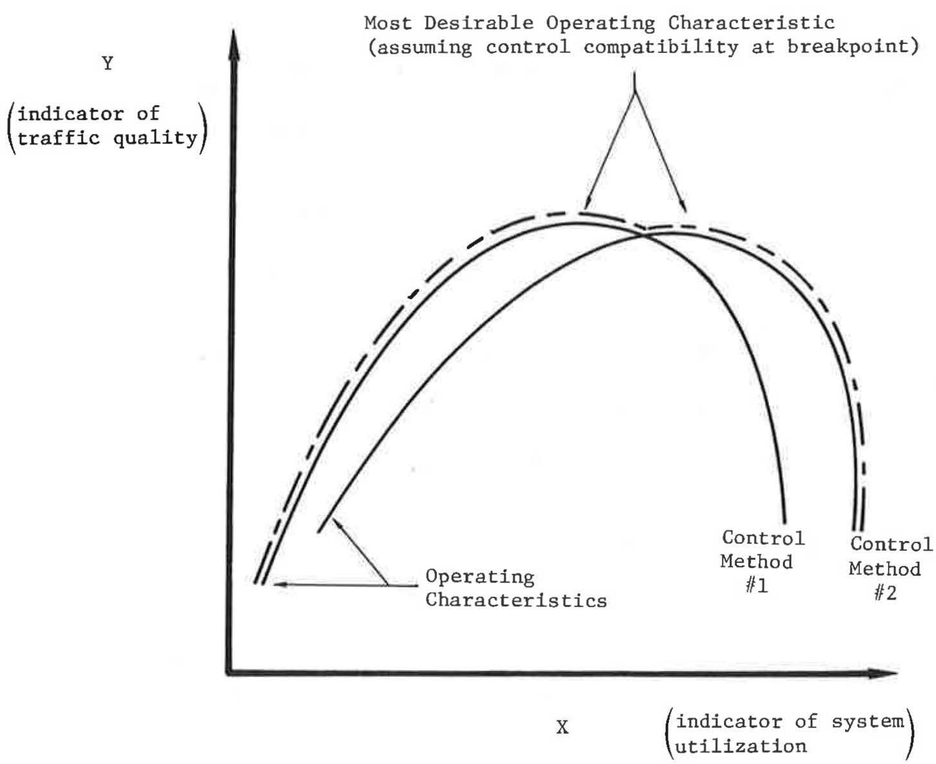

Figure 5. Selection of better control methods through UTCS operating characteristics.

of both how well the traffic is moving (travel time and energy ratio) and how much traffic is being moved (service rate). In on-line optimization, the amount of traffic in the system can be taken as an "input" so that it is necessary only to optimize the movement of traffic. Consequently, service rate need not be included in the payoff function, and the quantity J can serve as the basic optimization variable.

The function J for the complete network should be minimized subject to the constraint that the maximum value of J_i for any individual link (intersection) does not exceed a specified value. These statements are expressed mathematically as follows:
Find

$$\text{minimum } J = \alpha \times \overline{TT} - \beta \times \overline{\eta}_E$$

subject to

$$J_i \leq (J_i)_{max} \text{ for } i = 1, 2, \ldots, N$$

where

$J_i = \alpha_i \times \overline{TT} - \beta_i \times \overline{\eta}E_i$, and
N = number of links (intersections).

Numerous variations of this basic payoff function are possible through choices of α and β. For example, α can be made proportional to service rate and β inversely proportional. Thus, emphasis would be placed on minimizing travel time in heavy traffic and on maximizing flow smoothness during light traffic. In heavy congestion, it can be shown that J is directly proportional to delay.

The exact form of the payoff function should be chosen in conjunction with the optimization technique employed so as to obtain the most convenient mathematical formulation of the optimization problem.

CONCLUSIONS

As a result of the preceding discussions and evaluations, several recommendations for UTCS evaluation and control are in order. These are as follows:

1. System objectives—maximization of service and optimization of quality of service.
2. Measures of effectiveness—traffic movement MOE; travel time, both average per vehicle and total; flow smoothness MOE: energy ratio, ratio of effective to measured kinetic energies; system utilization parameter: service rate (total travel), product of output volume (count) and link length; and other parameters as desired by the traffic engineer.
3. Monitoring levels—data gathering on link (block-by-block) basis for real-time evaluation/control and off-line analysis; summaries on both system-wide and worst-element basis; and surveillance area extending beyond control area.
4. Required traffic parameters—free-flow spot speed (and time-tag), count, queue status, signal state, and arrival rates (major sources/sinks).
5. Operating characteristics for system evaluation—travel time ($\overline{TT}$ and/or TTT) versus service rate (R), energy ratio ($\overline{\eta}_E$) versus R, and $J = \alpha \times \overline{TT} - \beta \times \overline{\eta}_E$ versus R.
6. Real-time payoff function—minimize $J = \alpha \times \overline{TT} - \beta \times \overline{\eta}_E$ subject to $(\alpha_i \times \overline{TT}_i - \beta_i \times \overline{\eta}_{Ei}) \leq (J_i)_{max}$.

ACKNOWLEDGMENT

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Operational Efficiency Evaluation of Selected At-Grade Intersections

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The purpose of this study was to evaluate the operational efficiency of certain types of unsignalized priority-type intersections in terms of average waiting time and average number of vehicles that have to wait in the queue at the minor street approaches. Queuing theory techniques are employed in this study. Four intersections of varying geometrics are investigated. Utilizing the information on critical time gaps for the minor street traffic and peak-hour flows on the major street traffic, delays and queue lengths are calculated. These are then checked with the observed values. A fairly good correlation was found between the theoretical and observed values. The intersections are then rated according to their operational efficiency.

•BECAUSE URBAN TRANSPORTATION SYSTEMS are planned with emphasis on automobile travel, it becomes increasingly important to ensure the optimum operation of the systems. However, as a chain is only as strong as its weakest link, so the overall operation of a highway system is highly dependent on the operations in critical sections. Intersections in the case of a street system and interchanges in the case of a freeway system can be regarded as the weak links. One very important component of the system is the at-grade intersection. It is important that the traffic engineer know which control is best for a given intersection. However, only meager information is available concerning controls below the level of traffic signals. The purpose of this study is to evaluate the operational efficiency of priority-type unsignalized intersections of varying geometrics relative to traffic performance. Priority-type intersections occur where one of the intersecting streets is given a definite priority over other streets. The non-priority or the minor street for such intersection is controlled by either a stop or yield sign, thus ensuring that the vehicles on the street having the priority will suffer little or no delay. The intersections investigated in this study have the minor street approaches controlled by a stop sign.

At unsignalized intersections the arrival rates and individual drivers generally determine the manner of operation. The operational characteristics of these intersections are a function of traffic flow and driver behavior patterns. In simplest terms, at an intersection, one flow of traffic looks for gaps in the opposing flow of traffic. At priority intersections, because the traffic on the major street is given the priority, it is clear that the traffic on the minor street is usually looking for gaps.

OBJECTIVES

This study is an attempt to gain an understanding of the operational efficiency of priority-type unsignalized intersections of varying geometrics relative to traffic performance. It provides traffic and safety engineers a method for evaluating operational efficiency of certain types of unsignalized priority-type intersections in terms of (a) average time a vehicle on the critical minor street approach has to wait in the queue,

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as well as in the system, before it is able to merge with or cross the major street traffic stream, and (b) average number of vehicles on the critical minor street approach that have to wait to merge with or cross the major street traffic stream.

STUDY SITES AND DATA COLLECTION PROCEDURE

Site Selection

After consultations with the personnel of the Safety Section of the District of Columbia Department of Highways and Traffic, the following 4 intersections were selected for investigation: Seventh Street and Michigan Avenue, N. E. (Fig. 1); Eleventh and P Streets, N. W. (Fig. 2); Ninth and K Streets, N. W. (Fig. 3); and Twelfth and C Streets, N. E. (Fig. 4).

Each intersection seemed to have unusually high delay to the vehicles on the minor street approaches. All were unsignalized, at-grade priority-type intersections. A brief description of each intersection location is given in Table 1.

Figure 1. Seventh Street and Michigan Avenue, N. E., intersection.

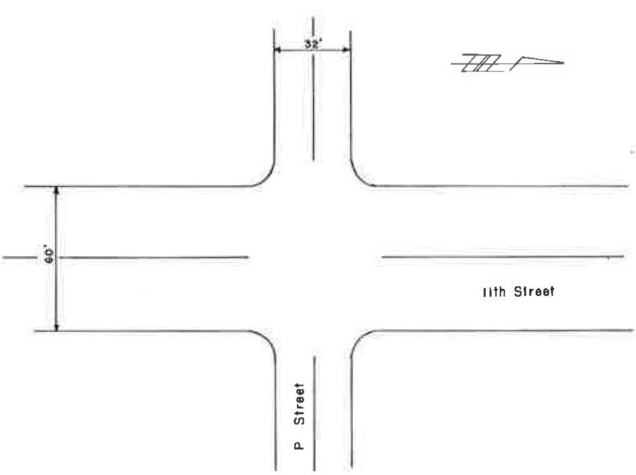

Figure 2. Eleventh and P Streets, N. W., intersection.

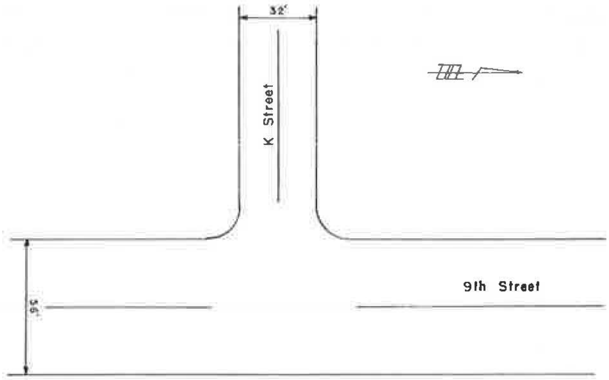

Figure 3. Ninth and K Streets, N. W., intersection.

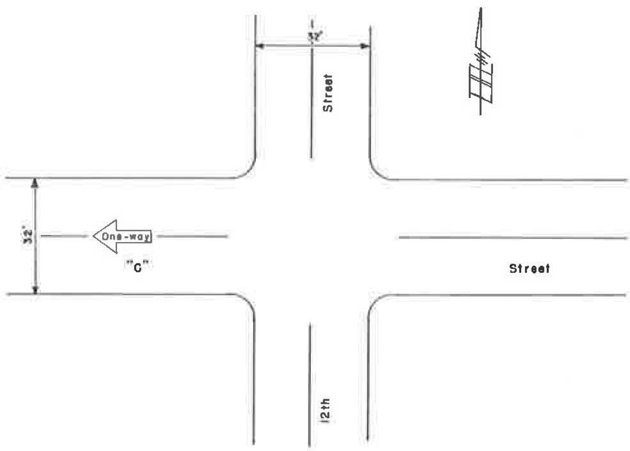

Figure 4. Twelfth and C Streets, N. E., intersection.

TABLE 1

SUMMARY OF STUDY INTERSECTIONS

Intersection	Type	Major Street	Minor Street	Location Within Metropolitan Area
Seventh Street and Michigan Ave., N.E.	4-legged, 2-way—2-way	Michigan Avenue	Seventh Street	Outlying business district
Eleventh and P Streets, N. W.	4-legged, 2-way—2-way	Eleventh Street	P Street	Outlying business district
Ninth and K Streets, N. W.	T-type, 2-way—2-way	Ninth Street	K Street	Fringe area
Twelfth and C Streets, N. E.	4-legged, C Street 1-way and Twelfth Street 2-way	C Street	Twelfth Street	Outlying business district

Data Collection for Peak-Hour Demand

One-hour traffic counts on a 5-minute basis were obtained for all approaches on each intersection study site during the afternoon peak period. Three recorders were found to be adequate for this purpose. Traffic counts for each approach were stratified into left-turn, through, and right-turn maneuvers.

Data Collection for Gap Measurements

To provide a permanent study record and to facilitate desired exactness of measurement, time-lapse photography was selected as the most appropriate means of recording gaps, lags, and headways.

The camera used was a 16-mm Bolex with a wide-angle lens on loan from the District of Columbia Department of Highways and Traffic. The camera was equipped with time-lapse apparatus and was set to take pictures at one-second intervals. Checks were made periodically during the data collection to ensure that the camera was operating accurately. Color movie film was used for ease of distinguishing individual vehicles.

Data collection was performed with the same procedure at all 4 study locations. At each intersection the camera was mounted on a tripod at some vantage point located near the side street approach. The camera was positioned about 50 ft from the main street to view the entire intersection area. A typical field installation is shown in Figure 5.

Data were collected on a weekday during the morning peak period. About 2 hours of filming was done for each intersection. Field data were collected only when the weather was clear and the pavements were dry. The speed of the camera was frequently checked by a stopwatch.

The developed film was viewed by a microfilm projector. The projector has a frame counter, and the film can be advanced or reversed one frame at a time. A stopped vehicle on the minor street approach leg either proceeded straight through the intersection, turned right, or turned left. If a driver went straight through the intersection, the path of movement intersected that of vehicles from both the right and the left. When a right turn was made, the vehicle merged with traffic coming from the left and did not conflict with the traffic from the right. On a left-turn maneuver the path of a main street vehicle approaching from the left was crossed, and the vehicle merged with the major street traffic coming from the right. The property lines on each approach leg were used as reference points to determine lags, gaps, and headways. It was possible to make measurements to the nearest half second.

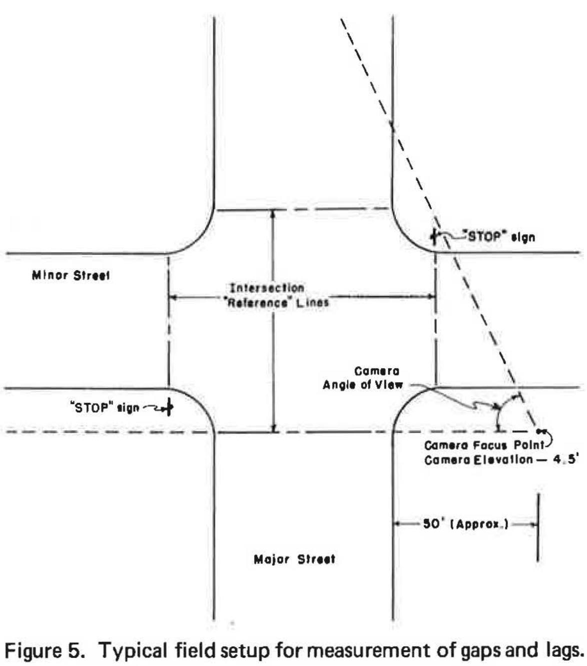

Figure 5. Typical field setup for measurement of gaps and lags.

OPERATIONAL EFFICIENCY ANALYSIS

Delay

This is a significantly important parameter because of its economic importance. In this study, the delay accruing to each vehicle is its idling time, or the time that the vehicle is not in motion. For all 4 intersections studied, the traffic at the minor street approaches was controlled by a stop sign whereas the major street traffic had no traffic control and always had the right-of-way. The only delay to the major street traffic was caused by a right- or left-turning maneuver. Every vehicle on the minor street approach leg had to come to a complete stop before entering the intersection.

The delay at an unsignalized intersection depends both on the physical layout of the intersection and on the volume of traffic on each approach. The physical layout affects the sight distance, which in turn affects the size of the acceptable lag or gap.

The delay due to traffic flow on each minor approach will increase as the traffic volume on the main route increases because the number of acceptable lags or gaps for entering the intersection will be reduced.

Headway Distribution

Several studies (1, 2, 3) have confirmed that the vehicle arrivals at intersection approaches conform to the Poisson law.

Therefore, in the theoretical derivations to obtain the total average delay to a minor street vehicle waiting to cross or merge with the major street traffic and to determine the average number of vehicles waiting on the critical minor street approach, it is assumed that the distribution of major street traffic arrivals is Poisson, i.e., that the probability that a given gap is between t and t + dt seconds is given by an expression of the form qe^{-qt} where q is the flow.

If the vehicle arrivals follow Poisson distribution, it follows that the headways between the vehicles are exponentially distributed. That is $P(h \geq t) = e^{-\lambda t}$ where λ = mean arrival rate in vehicles per time interval.

According to Adams (4), the proportion of time occupied by intervals greater than t seconds is given by the expression

$$P(h > t) = e^{-\lambda t} (\lambda t + 1)$$

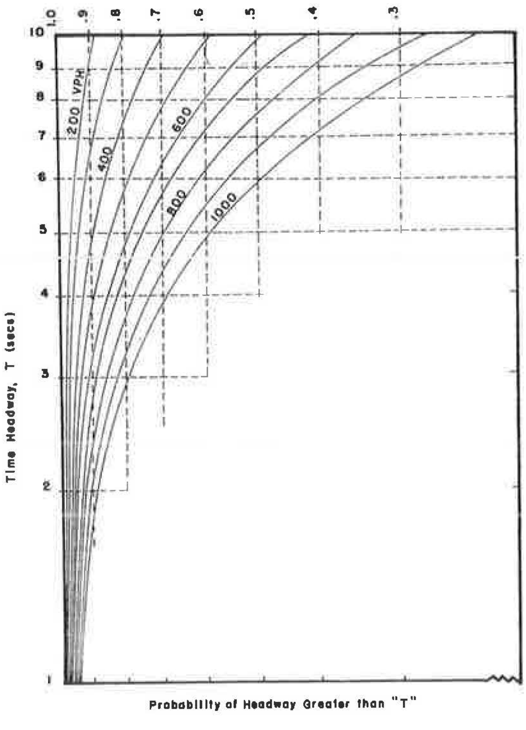

Figure 6. Cumulative probability curves for headway distribution—volume range 200 to 1,000 vph.

where λ = vehicles per second.

Conversely, the proportion of time occupied by intervals less than or equal to t is

$$P(h \leq t) = 1 - e^{-\lambda t} (\lambda t + 1)$$

In one hour the expected number of intervals greater than or equal to t is $Te^{-\lambda t}$ where T = vehicles per hour.

Similarly, the expected number of intervals greater than t + dt is

$$Te^{-\lambda(t + dt)} = Te^{-(\lambda t + \lambda dt)} = Te^{-\lambda t} e^{-\lambda dt}$$

by the rule for addition of indexes.

The cumulative frequency curves in Figures 6 and 7 show the theoretical relationship between gap availability and the critical gap, assuming that the distribution of headways on the main street is described by the exponential distribution.

Gap and Lag Acceptance

The acceptance or rejection of a time gap is a binomial response and is dependent on the size of the gap. The minimum time gap that a driver accepts is fixed for that driver. He will reject all gaps smaller

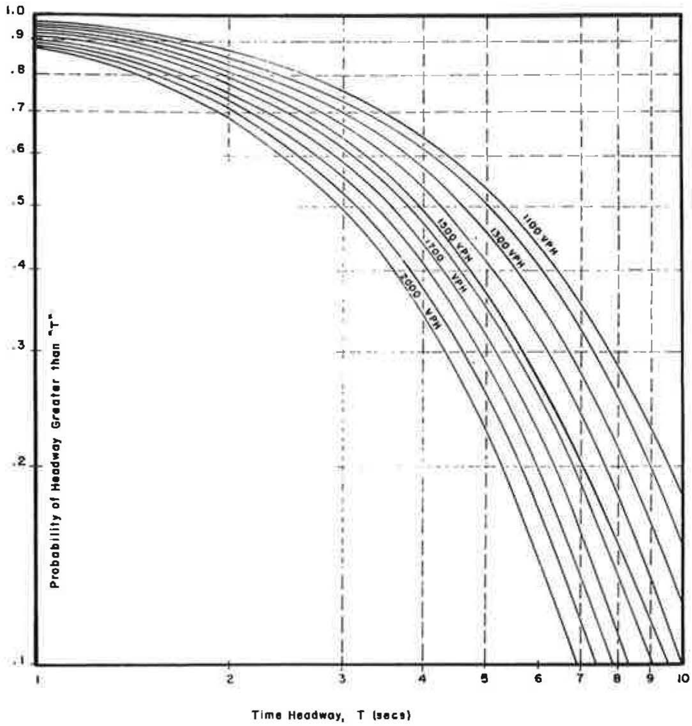

Figure 7. Cumulative probability curves for headway distribution—volume range 1,100 to 2,000 vph.

than that time interval and accept all gaps larger than that time gap. There is an evidence that this minimum acceptable time gap would decrease with time pressure and the number of vehicles in the major traffic stream.

A lag at an intersection may be defined as the time interval between the arrival of the minor street vehicle, and the arrival thereafter of the first vehicle on the major street at a reference point.

A gap at an intersection is defined as each time headway formed by successive crossings of a reference line by major street vehicles, regardless of direction of travel. If the minor street vehicle moves through the intersection before the arrival of the first major street vehicle, the driver of the minor street is said to "accept" the lag. If he remains until after the first vehicle passes, he has "rejected" the lag. After rejecting a lag, he then evaluates the gaps between the successive vehicles. Each gap that he fails to accommodate his vehicle into is said to be rejected. The gap that the driver finally moves into is said to be accepted.

Critical Gap or Lag

The method developed by Bissell (5) and Raff (6) to determine the value of a critical lag or gap for a minor street approach was used in this study. In this method, 2 cumulative distributions are plotted on the same graph. One curve describes the number of gaps or lags accepted shorter than a time interval, and the other shows the rejected number of gaps or lags longer than this interval. The value of the critical gap is the time at which the 2 curves intersect. A single value of critical gap was determined for left-turn and through movements for each study intersection. This was done after field observations indicated that there was no significant difference in the gap acceptance characteristics for these 2 maneuvers. A separate value of critical gap was determined for the right-turn maneuver for each study intersection. A typical graph showing the determination of critical time gap, T_c, is shown in Figure 8. The critical time gaps, T_c, for the selected intersections are given in Table 2. It is seen from the table that the critical time gaps are different for different intersections depending on the sight distance, traffic volume, and speed.

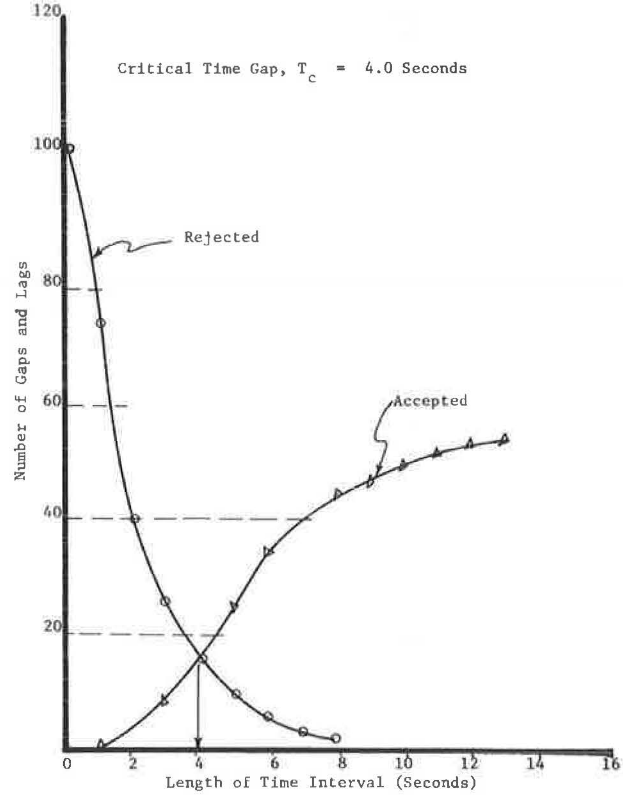

Figure 8. Distribution of accepted and rejected lags and gaps at Twelfth and C Streets, N. E., intersection, right-turn maneuvers.

Crossing and Merging Delays

Delay is useful in describing the level of service at an intersection or in a system of streets. Also it lends itself to economic analysis.

In an operation where a minor stream of traffic has to yield right-of-way to the major street traffic, it is important to determine the average time a vehicle has to wait before entering the intersection. The waiting driver measures each time gap, t, in the major street traffic until he finds an acceptable gap, T, that he believes to be of sufficient length to provide him a safe entry into an intersection. If he accepts the first gap (t > T), his waiting time is zero. If he rejects the first gap (t < T) and accepts the second gap, his waiting time is equal to one time gap interval. Queuing theory methods can be applied to establish the probability for the number of time gap intervals that a vehicle will have to wait before it can enter the intersection safely.

Thus we can write the probability, P_n, for the driver that he will have to wait for n intervals each less than T_c (critical time gap) seconds before entering the intersection as

$$P_n = (1 - \rho)\rho^n \text{ for } n > 1 \qquad (1)$$

where n = number of time intervals that a vehicle has to wait and ρ(traffic intensity) = λ/μ. We can also express the mean number of time intervals for which a vehicle has to wait as

$$E(n) = \frac{\rho}{1 - \rho}, \rho < 1 \qquad (2)$$

$$\rho = P(t < T_c) = \int_0^{T_c} f(t)dt \qquad (3)$$

TABLE 2
SUMMARY OF CRITICAL TIME GAPS

Intersection	Maneuver (sec) Right Turn	Maneuver (sec) Left Turn and Through
Seventh Street and Michigan Avenue, N. E.	5.5	7.0
Eleventh and P Streets, N. W.	5.5	7.0
Ninth and K Streets, N. W.	5.7	7.2
Twelfth and C Streets, N. E.	4.0	5.6

where f(t) is the density function of time gaps in the major traffic stream. Thus we have

$$E(n) = \frac{\rho}{1 - \rho} = \frac{\int_{0}^{T_c} f(t)\,dt}{\int_{T_c}^{\alpha} f(t)\,dt} \tag{4}$$

The average time a minor street vehicle will have to wait to find an acceptable gap in order to cross or merge with the major street traffic is the product of expected number of intervals less than T_c, E(n), and the mean length of time gap. The average length of time gap less than T_c is, in turn, equal to the total time less than T_c seconds divided by the number of time gaps less than T_c seconds.

$$\text{Average length of time gaps} < T_c = \frac{q \int_{0}^{T_c} t \times f(t)\,dt}{q \int_{0}^{T_c} f(t)\,dt} \tag{5}$$

where q is the rate of flow.

Multiplying Eqs. 4 and 5 yields the average waiting time for a minor street vehicle to merge or cross.

$$d = \frac{\int_{0}^{T_c} t \times f(t)\,dt}{\int_{T_c}^{\alpha} f(t)\,dt} \tag{6}$$

We recall that the percentage of minor street vehicles actually delayed is given by Eq. 2. It is evident that the average waiting time of those vehicles that are actually delayed is given by the expression

$$d' = \frac{d}{\int_{0}^{T_c} f(t)\,dt} \tag{7}$$

It should be remembered that the delays expressed in Eqs. 5 and 6 are for single vehicles approaching the minor street to merge or cross the main street.

Adams (4) and Tanner (8) have obtained theoretical derivations for vehicles attempting to merge or cross on the assumption that the major street traffic is exponentially distributed.

$$f(t) = qe^{-qt} \tag{8}$$

Substituting Eq. 7 in Eq. 5, the mean delay, d, for a critical time gap, T_c, and major street traffic flow, q, is

$$d = \frac{\int_{0}^{T_c} t \times qe^{-qt} \times dt}{\int_{T_c}^{\alpha} qe^{-qt} \times dt} = \frac{tq \int_{0}^{T_c} e^{-qt} \times dt}{q \int_{T_c}^{\alpha} e^{-qt} \times dt} \tag{9}$$

$$d = q^{-1}\left(e^{qT_c} - q^{T_c} - 1\right)$$

Equation 9 is shown in Figure 9, where delay, d, is shown as a function of major street volume, q, for various values of critical time gaps, T_c.

Determination of Total Delay to Minor Street Traffic

Our discussion up to now has been primarily concerned with average delay to the vehicle on the minor street and in a position to cross or merge with the major street traffic. It is important to determine how many vehicles on the average are waiting on the minor street to enter the intersection and also average waiting time for a queue of minor street vehicles waiting to merge or cross major street traffic. We can also use these 2 values as figures-of-merit to evaluate the operational efficiency of intersections.

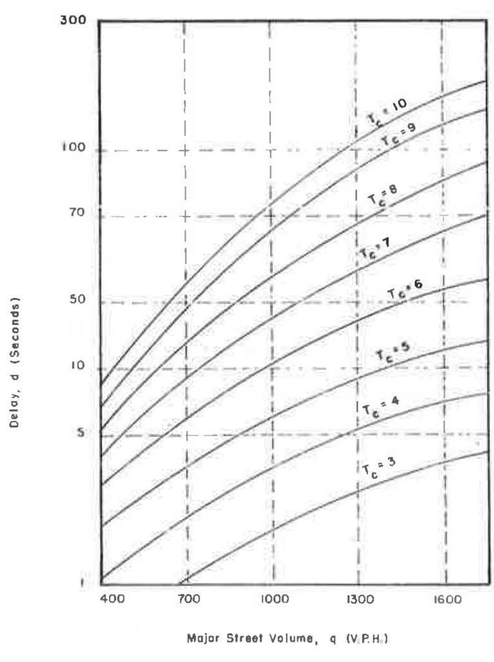

Figure 9. Delay, d, as a function of major street volume, q, for various values of critical time gaps, T_c (seconds).

In our case, a sequence of vehicles arrives at the minor street approach legs to merge or cross the major street traffic. If and when there is a suitable lag or gap available in the major street traffic, a vehicle from the minor street is able to enter the intersection, and thus a minor street vehicle is discharged. Usually there are variations in the regularity of vehicle arrivals or in the waiting time required to "put through" a vehicle from the minor street, or both; hence, there will be fluctuations in all aspects of intersection operation. More vehicles may arrive on the minor street than the major street can immediately accommodate, and a queue of varying length will be formed. The variables associated with the operation will be stochastic variables and will fluctuate with time above and below some average value or rate. Morse (9) explains that instead of trying to predict in detail how the state of the system changes with time, we can calculate the probabilities that the system is in each of the possible states. These probabilities facilitate calculations of average queue length and average time that the vehicle has to wait in the system.

The state of the intersection operation is dependent on the average arrival rate on the minor street, λ, and the average service rate, μ, on the major street. The traffic intensity factor, ρ, is the ratio of the vehicle rate, λ, and service rate, μ, and represents the fraction of the time the service channel is used; i.e., either a merging or crossing operation from the minor street traffic can be performed. If $\rho > 1$, there is no "steady-state" solution, and arriving vehicles join an ever-increasing queue; if $\rho < 1$, steady-state solutions can be determined if λ and μ are known.

Let q_a be the average rate of arrival on a critical approach of the minor street. The minor street approach with the highest arrival rate is considered as the critical approach. The vehicles arriving on the critical approach must stop, and the main street traffic has the right-of-way. They form a single-lane queue waiting for a vehicle at the head of the queue to merge with or cross the major street traffic. For a Poisson distribution of arrivals and exponential distribution of service rate,

$$\text{Expected number of units in the system, } E(n) = \frac{\lambda}{\mu - \lambda} \tag{10}$$

$$\text{Expected number of units in the queue, } E(m) = E(n) - \rho \tag{11}$$

$$\text{Expected waiting time in the system, } E(\gamma) = \frac{1}{\mu - \lambda} \tag{12}$$

$$\text{Expected waiting time in the queue, } E(w) = E(\gamma) - \frac{1}{\mu} \tag{13}$$

Major and Buckley (10) have interpreted the service time for the queue, μ, as identical to the summation of the rejected lags and gaps for a minor street approach vehicle in position to merge. This is the same as the delay, d (Eq. 9, Fig. 9), in this study. Based on the assumption that the average service time is equal to the average merging delay and the distribution of service times is of the exponential form, the peak-period operational efficiency of each study intersection is obtained in terms of queue lengths and waiting times at the minor street approaches.

1. Seventh Street and Michigan Avenue, N. E. intersection, left-turn and through maneuvers, critical approach at Catholic University entrance.

$$\text{Arrival rate} = 81 \text{ vph} = \frac{81}{3{,}600} = 0.0225 \text{ vps}$$

$$\text{Service rate} = 35 \text{ sec/vehicle}$$

(from Fig. 9, for critical time gap, T_c, equal to 7 sec, and major street traffic flow, q, equal to 1,450 vph) or

$$\frac{3{,}600}{35} = 103 \text{ vph} = \frac{1}{35} = 0.0286 \text{ vps}$$

$$\rho(\text{traffic intensity}) = \frac{\lambda}{\mu} = \frac{0.0225}{0.0286} = 0.7867$$

$$\text{Expected time in the system, } E(\gamma) = \frac{1}{\mu - \lambda} = \frac{1}{0.0286 - 0.0225} = \frac{1}{0.0061} = 164 \text{ sec}$$

$$\text{Expected time in the queue, } E(w) = E(\gamma) - \frac{1}{\mu} = 164 - \frac{1}{0.0286} = 164 - 35 = 129 \text{ sec}$$

$$\text{Expected number in the system, } E(n) = \frac{\lambda}{\mu - \lambda} = \frac{0.0225}{0.0286 - 0.0225} = \frac{0.0225}{0.0061}$$

$$= 3.69 \text{ vehicles}$$

$$\text{Expected number in the queue, } E(m) = E(n) - \frac{\lambda}{\mu} = 3.69 - 0.79 = 2.90 \text{ vehicles}$$

2. Seventh Street and Michigan Avenue, N. E. intersection, right-turn maneuver, critical approach at Seventh Street.

$$\lambda(\text{arrival rate}) = 141 \text{ vph} = \frac{141}{3{,}600} = 0.0392 \text{ vps}$$

$$\mu(\text{service rate}) = 15 \text{ sec/vehicle}$$

(from Fig. 9, for critical time gap, T_c, equal to 5.5 sec, and major street traffic flow, q, equal to 1,450 vph) or

$$\frac{3{,}600}{15} = 240 \text{ vph} = \frac{1}{15} = 0.0667 \text{ vps}$$

$$\rho(\text{traffic intensity}) = \frac{\lambda}{\mu} = \frac{0.0392}{0.0667} = 0.5877$$

$$\text{Expected time in the system, } E(\gamma) = \frac{1}{\mu - \lambda} = \frac{1}{0.0667 - 0.0392} = \frac{1}{0.0275} = 36.36 \text{ sec}$$

$$\text{Expected time in the queue, } E(w) = E(\gamma) - \frac{1}{\mu} = 36.36 - \frac{1}{0.0667} = 36.36 - 14.99$$

$$= 21.37 \text{ sec}$$

$$\text{Expected number in the system, } E(n) = \frac{\lambda}{\mu - \lambda} = \frac{0.0392}{0.0667 - 0.0392} = \frac{0.0392}{0.0275}$$

$$= 1.43 \text{ vehicles}$$

$$\text{Expected number in the queue, } E(m) = E(n) - \frac{\lambda}{\mu} = 1.43 - 0.5877 = 0.842 \text{ vehicles}$$

In a similar manner waiting times and queue lengths were determined for other intersections.

In order to evaluate the reliability of the theoretically calculated values of queue lengths and waiting times for the vehicles on the minor street approaches, it was decided to obtain field measurements.

The expected time in the system and in the queue are the average waiting times for a vehicle on the critical minor street approach. It would be almost impossible to obtain waiting times for each vehicle approaching the critical minor street approach; hence a systematic random sampling technique was used. The method consisted of selecting every fifth vehicle at a critical minor street approach and recording its waiting time in the queue and in the system. The waiting times for left-turn and through maneuvers were recorded as one group, and right-turn maneuvers as another group. The data were collected at each location for a period of one hour during the peak period. The mean waiting time was determined from the sample.

The field measurements to obtain the mean number of vehicles waiting in the queue and in the system at the minor street approach were obtained in a similar manner. In this case the number of vehicles in the queue and in the system were observed at exactly 3-minute intervals for a period of one hour during the peak period at each study intersection. Again, the queue lengths were obtained separately for left-turn and through maneuvers in one group and right-turn maneuvers in another group.

The results are given in Tables 3 and 4. It is seen that there is a fairly good correlation between the theoretical and observed values.

Operational Efficiency Evaluation

Data given in Tables 3 and 4 demonstrate the operational efficiency of 4 priority-type at-grade intersections with varying geometrics.

TABLE 3

EXPECTED AND OBSERVED WAITING TIMES AND NUMBER OF VEHICLES IN THE SYSTEM AND QUEUE FOR LEFT-TURN AND THROUGH MANEUVERS

Intersection	Expected Time in the System (sec)		Expected Time in the Queue (sec)		Expected Number in the System (vehicles)		Expected Number in the Queue (vehicles)	
	Theoretical	Observed	Theoretical	Observed	Theoretical	Observed	Theoretical	Observed
Seventh Street and Michigan Avenue, N. E.	164.00	190	129.00	143	3.69	5	2.90	3
Eleventh and P Streets, N. W.	277.78	262	247.75	260	8.25	7	7.358	6
Ninth and K Streets, N. W.	86.95	81	58.91	60	2.104	2	1.426	1
Twelfth and C Streets, N. W.	6.99	9	1.0	2	0.165	0	0.024	0

TABLE 4

EXPECTED AND OBSERVED WAITING TIMES AND NUMBER OF VEHICLES IN THE SYSTEM AND QUEUE FOR RIGHT-TURN MANEUVERS

Intersection	Expected Time in the System (sec)		Expected Time in the Queue (sec)		Expected Number in the System (vehicles)		Expected Number in the Queue (vehicles)	
	Theoretical	Observed	Theoretical	Observed	Theoretical	Observed	Theoretical	Observed
Seventh Street and Michigan Avenue, N. E.	36.36	42	21.37	25	1.43	2	0.842	1
Eleventh and P Streets, N. W.	15.38	13	2.38	3	0.183	0	0.028	0
Ninth and K Streets, N. W.	80.00	94	68	75	5.66	7	4.810	6
Twelfth and C Streets, N. E.	2.56	4	0.06	0	0.025	0	0.011	0

The Twelfth and C Streets, N. E., intersection is the most efficient one, both with respect to delay to the vehicles and the number of vehicles that have to wait in line on the minor street approaches. C Street, which is one-way, carries the major street traffic, whereas Twelfth Street is a 2-way minor street approach. The average waiting time for a left-turning or through vehicle on the critical minor street approach is about 7 seconds. The average waiting time in the queue is about one second. There is hardly any queue formed on the minor street approach. A right-turning vehicle has to wait about 3 seconds and has almost no waiting time in the queue. On the average, no queues are formed on the minor street approaches.

The Ninth and K Streets, N. W., intersection is the second most efficient for left-turn maneuvers from the minor street approach. However, it is the least efficient for right-turn maneuvers from the minor street approach. This is a T-type intersection. Ninth Street is a 2-way major street, and K Street is a single minor street approach. The average waiting time for a left-turn maneuver from the minor street approach is about 87 seconds, whereas average waiting time in the queue is about 59 seconds. The mean number of vehicles waiting to make a left-turn maneuver is two and about one in the queue. The average time a right-turning vehicle has to wait on the minor street approach is 80 seconds, which is almost the same as a left-turn maneuver. The average waiting time in the queue is 68 seconds, which is slightly more than the average waiting time in the queue for the left-turn maneuvers. The average number of vehicles on the minor street approach waiting to make a right-turn maneuver is about six, and the average number of vehicles waiting in the queue to make the right-turn maneuver is about five. Both the queue length and the number of vehicles on the minor street approach waiting to make the right-turn are considerably higher than for the left-turn maneuver. It is noted here that this intersection is least efficient operationally for right-turn maneuvers.

The Seventh Street and Michigan Avenue, N. E., intersection rates the third most efficient for all maneuvers from the minor street approaches. This is a 4-legged, 2-way intersection. However, this is a unique intersection because the minor street approaches are offset by about 100 ft. Also one approach is at a right angle and the other is at about 60 deg to the major street. The average delay to the left-turn and through vehicle on the critical minor street approach is about 164 seconds, and the waiting time in the queue is about 129 seconds. The mean number of vehicles in the system is about four and the number of vehicles in the queue is about three. For right-turn maneuvers, however, the waiting times and the queue lengths are considerably shorter. The mean waiting time in the system and in the queue on the critical minor street approach is about 36 and 21 seconds respectively. The average number of vehicles in the queue and in the system on the critical minor street approaches is about two and one respectively.

The Eleventh and P Streets, N. W., intersection is a typical 4-legged, 2-way—2-way intersection. It is the least efficient for left-turn and through maneuvers, whereas the second most efficient for right-turn maneuvers. The average delay to a vehicle on the critical minor street approach is about 277 seconds, and the time spent in the queue is about 248 seconds. The average number of vehicles in the system and in the queue on the critical minor street approach is eight and seven respectively. The unusually high waiting time and queue formation is due to a considerably high traffic demand on the major street during the peak periods as well as existing moderate demand on the minor street. For right-turn maneuvers, the intersection is the second most efficient. The waiting times in system and in the queue are about 15 and 3 seconds respectively, and queues are seldom formed.

SUMMARY AND CONCLUSIONS

Summary

This investigation has shown how queuing theory techniques can be employed to evaluate the operational efficiency of priority-type at-grade intersections in terms of waiting times in the system and in the queue at the minor street approaches. It also indicates the number of vehicles that have to wait in the system and in the queue at the minor street approaches.

The applicability of this method has been tested for 4 selected priority-type intersections of varying geometrics of a large metropolitan area.

CONCLUSIONS

1. There were some indications that the driver on the minor street tends to accept a shorter gap under pressure (i.e., when a queue of 2 or more vehicles is formed behind him) than the gap he would accept under normal conditions.

2. There was a fairly good correlation between the theoretical and observed values of the mean waiting time in the system and in the queue and of the mean number of vehicles waiting in the system and in the queue for the vehicles on the minor street approaches. The operational efficiency in terms of delay and queue lengths is given in Table 5 in the descending rank order for left-turn and through maneuvers and for right-turn maneuvers.

TABLE 5

RANK OF INTERSECTION BY OPERATION EFFICIENCY

Intersection	Left-Turn and Through Maneuvers From the Minor Street Approaches	Right-Turn Maneuvers From the Minor Street Approaches
Twelfth and C Streets, N. E.	1	1
Ninth and K Streets, N. W.	2	4
Seventh Street and Michigan Avenue, N. E.	3	3
Eleventh and P Streets, N. W.	4	2

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The Environmental Influence of Rain on Freeway Capacity

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The capacity of a freeway is defined by physical factors of the roadway, traffic factors, and environmental disturbances. Physical characteristics of the roadway are fixed by design and construction and exert a constant influence on freeway capacity. Traffic factors are variable in their effect on freeway capacity, but they are also subject to control and regulation to some degree. Environmental factors cannot be controlled, and thus only their effect can be compensated for by preparation in advance of occurrence. Rain, the most common environmental disturbance to capacity, was studied in this research. Rainfall information for March 1968 through December 1968 in Houston, Texas, was correlated with traffic data records of the Gulf Freeway Surveillance and Control Center operation to obtain data indicating the effect of rain on freeway capacity. Rain was found to reduce the capacity of a freeway section to between 81 and 86 percent of the dry weather capacity with 95 percent confidence.

•CAPACITY OF A FREEWAY SECTION is a function of numerous variables. These variables can be classified into 3 groups defined by the roadway subsystem, driver-vehicle subsystem, and the environment of the highway operating system. Physical factors, related to the roadway subsystem, include lane width, horizontal curvature, grade, and condition of pavement. Traffic factors, related to the driver-vehicle subsystem, include composition of traffic stream, driver characteristics, and vehicular capabilities. Environmental factors (influencing capacity but not related to elements subject to control by design or operation) include light intensity, rain, fog, ice, and snow. Once a highway is constructed, the physical factors influencing capacity assume a constant value until reconstruction is initiated. The influence of traffic factors on capacity is subject to fluctuation as the characteristics of the traffic vary. To some extent, the effect of traffic factors on capacity can be reduced through regulation and control. Although it is possible through proper design to minimize the effect of environmental disturbances, no control can be exercised over their occurrence.

The effect of the physical and traffic factors on capacity have been extensively investigated and documented (1). Moskowitz and Newman reported in 1963 that the effects of weather and lighting were not treated at all in their research on freeway capacity and that this represented a deficiency in knowledge at that time (2). A survey of technical literature indicates that little has been done to fill this void in knowledge.

Design and/or control of a freeway may be based on normal environmental conditions. However, to have a comprehensive (system) design or control plan, the operation must be predictable under degraded environmental conditions. Therefore, this paper reports research undertaken to evaluate the effect of the most common environmental disturbance, rain, on the primary freeway operation parameter, capacity.

Paper sponsored by Committee on Highway Capacity.

RESEARCH METHODOLOGY

Study Site

Data were collected on the Gulf Freeway (I-45) in Houston, Texas. This facility was appropriate because it was available for research study; it had a fully operational freeway control system and an automatic detection system interconnected to a digital computer for data acquisition needs. The research was conducted by the Texas Transportation Institute for the Texas Highway Department in cooperation with the U. S. Department of Transportation.

Data Collected

The 3-lane, inbound portion of the Gulf Freeway from state highway 225 to the Houston central business district is divided into 4 subsystems as shown in Figure 1. Loop detectors, represented by dots in the figure, are located on all ramps and on the freeway lanes to define subsystem 2 (SS2) through subsystem 5 (SS5). A digital computer monitors these detectors for inputs to establish real-time freeway control during both the a.m. and p.m. peak-period flows. The computer also simultaneously accumulates traffic count information at each detector that can be converted into traffic flow and subsystem density measurements. Flow and density were recorded each minute for each of the 4 closed subsystems shown in Figure 1 during the period from 6:30 a.m. to 8:30 a.m. The one-minute traffic data collected for each subsystem in the 3.5-mile freeway section were converted into 5-minute flow rates and expressed as vehicles per hour (vph) across all 3 lanes. Average density was calculated in terms of vehicles per mile (vpm) for all 3 lanes in each subsystem. A typical sample of the data collected for one day is given in Table 1.

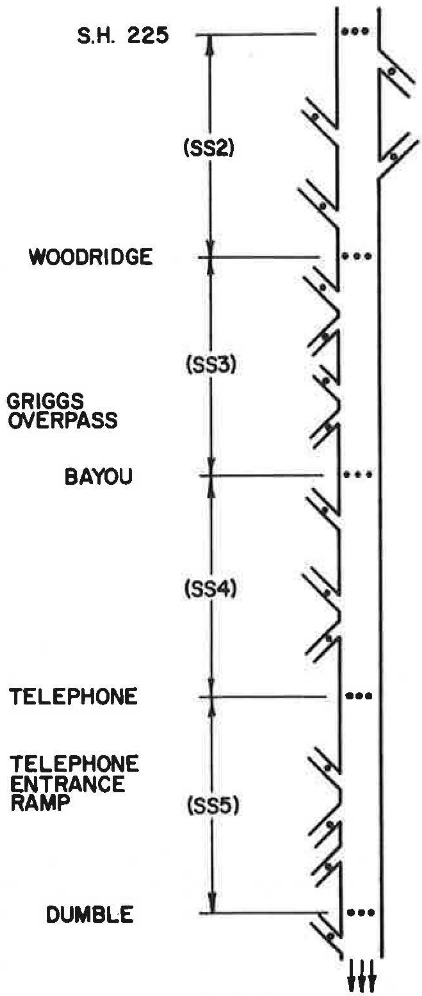

Figure 1. Location of vehicle detectors on inbound Gulf Freeway

Rainfall records were obtained from the U. S. Weather Bureau at Houston's William P. Hobby Airport and in downtown Houston. The Hobby Airport weather station is 4 miles southeast of the Freeway Surveillance and Control Center and the downtown Houston station is 4 miles northwest of the control center. Daily logs of observed weather conditions were also made at the Freeway Surveillance and Control Center.

The data did not provide information on rainfall intensity applicable to this study because the rainfall rate could vary throughout the length of the 3.5 miles of freeway and throughout the 2-hour peak period. Each day was simply classified as either "dry" or "rain." If unstable or inconsistent weather conditions existed during the peak period, or if no distinct classification was possible for a particular day, that data sample was discarded.

Capacity-Demand Considerations

The data collected on any given day will have a variation in the flow during the peak period. However, the maximum flow attained during the peak period on any given day is not always the capacity. The demand on a section must be great enough to exceed

TABLE 1

FLOW RATES AND DENSITIES ON THE INBOUND GULF FREEWAY, JUNE 25, 1968

5 MIN PERIOD ENDING	VPH AT 225	DEN IN SS2	VPH AT WOODRG	DEN IN SS3	VPH AT OVERPS	VPH AT BAYOU	DEN IN SS4	VPH AT TELEPH	DEN IN SS5	VPH AT MERGE	VPH AT CUMBLE	DATE
5	2184	18	2916	55	2256	1920	30	1296	19	1320	516	[illegible]
10	2304	22	3792	71	3792	3048	47	3576	56	3636	3216	68
15	2412	28	4320	83	4548	3888	56	3828	66	4032	3168	68
20	2844	27	4692	95	4668	4080	62	4272	78	4380	3552	2568
25	3612	46	5340	107	5220	4452	68	4728	95	4896	4116	62568
30	3600	49	5760	149	5844	5232	82	5544	118	5736	4392	62568
35	3852	54	5076	151	5388	4824	82	5352	119	5760	4704	62568
40	4200	62	5424	151	5688	5124	89	5304	112	5496	4656	62568
45	3816	70	5040	150	5664	5112	102	5472	131	5712	4920	62568
50	4152	71	5208	154	5736	5316	99	5640	138	5916	5004	62568
55	4452	78	5040	143	5808	5280	94	5760	160	6240	5364	62568
60	4488	94	4956	171	5496	5124	94	5376	158	5904	5376	62568
65	3780	101	4644	178	5700	5352	102	5460	171	5916	5124	62568
70	3516	104	4704	157	5472	5100	123	5184	186	5556	5232	62568
75	3816	102	4620	165	5220	4884	146	5208	192	5604	4992	62568
80	3588	115	4440	176	4980	4536	165	4776	176	5124	4992	62568
85	3264	110	4548	202	4812	4488	162	5076	177	5472	4668	62568
90	3948	112	3972	206	5208	4788	155	5208	189	5508	4764	62568
95	3228	99	4908	188	5316	4812	161	4896	183	5256	4800	62568
100	3804	91	4644	187	5184	4692	159	5280	178	5640	4932	62568
105	3180	84	4104	177	5196	4704	138	5208	175	5568	4656	62568
110	3480	70	4716	158	5160	4680	135	4944	192	5280	4464	62568
115	3324	42	5028	172	5136	4476	151	5052	181	5328	4392	62568
120	3732	46	4416	167	5256	4620	128	5472	182	5736	4740	62568
125	3264	39	5004	152	5316	4620	118	5316	178	5556	4392	62568
130	3374	30	4728	141	5304	4608	102	5028	197	5244	4080	62568
135	3264	28	4488	99	4884	4404	110	4920	171	5016	4320	62568
140	168	1	960	19	2064	1728	39	3576	109	3732	4536	62568

the capacity. To avoid collecting volumes of unnecessary data, the historical back-log of information on the Gulf Freeway operation was utilized in this study. Two bottlenecks were selected for this analysis. The first bottleneck, denoted Griggs overpass, is located in subsystem 3 (SS3), and the second, identified as the Telephone merge, is located in subsystem 5 (SS5).

Definition of Capacity

For a consistent basis of comparison among the days for which traffic data were determined to be suitable on the basis of weather conditions, a means of identifying the capacity for the total subsystem environment had to be established. Even with a known bottleneck in the subsystem under consideration, it would have been possible to measure demand as a maximum flow rather than capacity. It was decided to define the capacity of the freeway associated with the peak-period sample as the maximum ordinate of the best-fit flow-density curve.

For each of the 2 subsystems, a pair of values representing the flow rate, q, and the density, k, was available for each 5 minutes of operation. The first 2 and the last 2 pairs of data points were deleted to avoid bias in the sample while the computerized

TABLE 2

TRAFFIC MODEL FIT FOR SUBSYSTEM 3, JUNE 25, 1968

EN= -1.00	RSMS= 2.755	DJ= 432.26	UF=********	QM= 5408.11	A-LEVEL= 0.578729	RATIO= 1.081
EN= -0.80	RSMS= 2.597	DJ= 405.97	UF= 380.78	QM= 5418.16	A-LEVEL= 0.649487	RATIO= 1.079
EN= -0.60	RSMS= 2.461	DJ= 384.75	UF= 210.70	QM= 5429.86	A-LEVEL= 0.709336	RATIO= 1.076
EN= -0.40	RSMS= 2.349	DJ= 367.23	UF= 153.99	QM= 5442.35	A-LEVEL= 0.757408	RATIO= 1.074
EN= -0.20	RSMS= 2.261	DJ= 352.54	UF= 125.60	QM= 5455.39	A-LEVEL= 0.793474	RATIO= 1.071
EN= 0.00	RSMS= 2.197	DJ= 340.04	UF= 108.55	QM= 5468.73	A-LEVEL= 0.818542	RATIO= 1.069
EN= 0.20	RSMS= 2.158	DJ= 329.30	UF= 97.17	QM= 5482.26	A-LEVEL= 0.833101	RATIO= 1.065
EN= 0.40	RSMS= 2.142	DJ= 319.96	UF= 89.02	QM= 5495.86	A-LEVEL= 0.838861	RATIO= 1.063
EN= 0.60	RSMS= 2.151	DJ= 311.78	UF= 82.90	QM= 5509.40	A-LEVEL= 0.835471	RATIO= 1.061
EN= 0.80	RSMS= 2.185	DJ= 304.56	UF= 78.11	QM= 5522.80	A-LEVEL= 0.823025	RATIO= 1.059
EN= 1.00	RSMS= 2.243	DJ= 298.15	UF= 74.27	QM= 5536.05	A-LEVEL= 0.800473	RATIO= 1.056

OPTIMUM (CLOSEST FITTING MODEL)

EN= 0.40	RSMS= 2.142	DJ= 319.96	UF= 89.02	QM= 5495.87	A-LEVEL= 0.838941	RATIO= 1.063

Note: EN = n value used in calculations; RSMS = residual mean square (small value indicates a better fit); DJ = jam density; UF = free speed (theoretically it is infinite when n = -1); QM = maximum ordinate of Eq. 1 or calculated capacity; A-LEVEL = an acceptance level; and RATIO = maximum observed 5-minute flow rate to QM.

counting system was starting and stopping. From 24 to 30 pairs of flow-density values remained for each subsystem per record day.

The flow-density model chosen was a generalized traffic flow model as given in Eq. 1 (3). The detailed development of the model is provided in the cited reference.

$$q = k \cdot u_f \left[1 - \left(k/k_j\right)^{(n+1)/2}\right] \qquad n > -1 \tag{1}$$

The corresponding relation for traffic stream speed is given by Eq. 2.

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

The model is a 3-parameter model: the exponent constant, n; the free speed, u_f; and the jam density, k_j. As indicated in Eqs. 1 and 2, n is restricted to a value greater than -1. Because the traffic flow data yielded only flow-density points, a method of establishing the appropriate parameters was established as follows.

The space-mean speed, u, corresponding to each pair of flow-density data points was calculated as

$$u = q/k \tag{3}$$

which produced pairs of speed-density points. A value of n was selected an an initial value, and a least squares regression was performed with Eq. 2 to give the value of u_f and k_j for the best fit to a particular day's data sample. The residual mean square of the fitted curve to the data was calculated. A second value of n was selected and the least squares regression repeated. If the second value of n provided a better fit to the data, as indicated by a smaller residual mean square, that value of n was selected over the previous one. The process was continued using a Fibonacci search until the optimum value of n was established (4).

Substituting the values of n, u_f, and k_j thus determined in Eq. 1 provides the optimum flow-density model. The maximum ordinate (flow) of this relation is considered to be the freeway capacity for a particular subsystem for the sampled day. The ratio of the highest observed 5-minute flow rate to the calculated capacity was noted and used for subjective evaluation of the procedure.

Table 2 gives an example of fitting the traffic model to the data collected on June 25, 1968, in subsystem 3. The acceptance level is the probability of obtaining the observed data sample from a process described exactly by the flow model used. This probability is the level of confidence at which the model would be accepted. The level at which the model was considered to be acceptable was 10 percent.

For the calculations given in Table 2, the optimum flow model would be

$$q = k \cdot 89.02 \left[1 - (k/319.96)^{0.7}\right] \tag{4}$$

or in terms of speed

$$u = 89.02 \left[1 - (k/319.96)^{0.7}\right] \tag{5}$$

Figure 2 shows an example of a speed versus density plot with both the observed data and the optimum model curve shown. Figure 3 shows an example of flow rate versus density with both observed data and the optimum curve shown. Both Figure 2 and Figure 3 are for the June 25, 1968, sample in subsystem 3.

Acceptability of Data Samples

The results of traffic flow model fitting and analysis of acceptability of model fit are given in Table 3 for all 24 peak-period samples available. All models that fit with an acceptance level greater than 10 percent were rated "accepted" and designated with an A in the table. All samples for which the model gave only an acceptance level of 1 to 10 percent were rated "doubtful" and designated with a D. Only 11 sample days for

TABLE 3

CAPACITIES AND MODEL ACCEPTANCE LEVELS

Date	Dry/Wet	Subsystem 3		Subsystem 5	
		Capacity[a] (vpm)	Acceptance Level	Capacity[a] (vpm)	Acceptance Level
Feb. 13	Wet	4,795 D	0.0772	4,817 U	0.0000
Mar. 21	Wet	5,000 U	0.0079	4,932 U	0.0001
May 6	Dry	5,836 D	0.0404	5,917 A	0.2903
May 7	Dry	5,541 A	0.2340	5,883 A	0.2342
May 8	Wet	5,338 U	0.0000	5,792 D	0.0350
May 9	Dry	5,684 D	0.0422	5,688 D	0.0180
June 6	Dry	5,640 A	0.5199	5,933 A	0.8764
June 12	Dry	5,732 A	0.3795	5,883 A	0.9195
June 21	Wet	4,530 A	0.9494	4,685 A	0.8022
June 25	Dry	5,495 A	0.8388	5,892 A	0.9653
June 26	Wet	4,729 A	0.9602	4,733 D	0.0542
Aug. 14	Dry	5,554 U	0.0002	5,710 A	0.1134
Sept. 12	Dry	5,652 U	0.0000	5,762 D	0.0241
Sept. 23	Wet	5,140 D	0.0872	5,359 D	0.0702
Oct. 17	Dry	5,380 U	0.0000	5,520 U	0.0000
Oct. 21	Dry	5,284 A	0.8282	5,689 A	0.9901
Oct. 29	Dry	5,711 U	0.0025	5,853 A	0.5382
Oct. 30	Dry	5,340 U	0.0089	5,619 U	0.0003
Oct. 31	Dry	5,641 A	0.4463	5,708 U	0.0000
Nov. 5	Dry	5,639 A	0.1961	5,829 U	0.0000
Nov. 6	Dry	5,592 A	0.1340	12,092 U	0.0000
Nov. 7	Dry	5,595 U	0.0001	5,675 U	0.0000
Nov. 8	Wet	5,208 U	0.0000	189,554 U	0.0000
Dec. 12	Wet	4,770 A	0.9759	4,995 A	0.2576

[a]A denotes acceptable; D, doubtful; U, unacceptable.

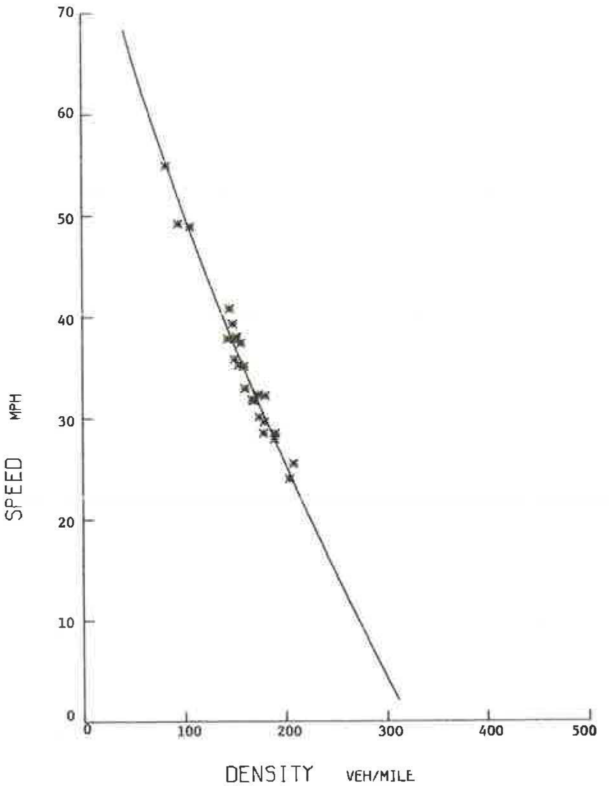

Figure 2. Speed-density relationship for subsystem 3, June 25, 1968.

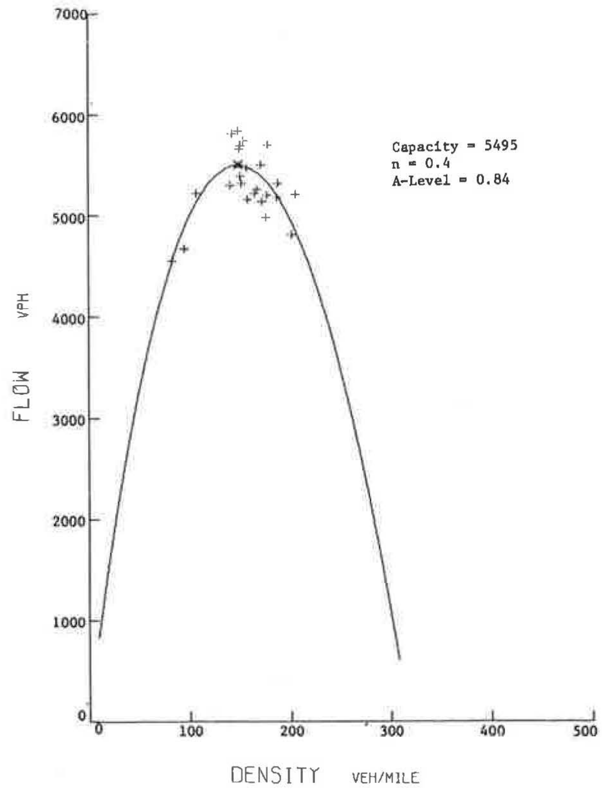

Figure 3. Flow-density relationship for subsystem 3, June 25, 1968.

subsystem 3 and 10 sample days for subsystem 5 were considered acceptable out of the original set of 24 sample days.

The high rate of disqualification of data samples may appear to indicate an inappropriate mathematical and statistical technique. However, this is not the case. The model developed was intended to provide a prediction of the capacity when only rain was a disturbance or when the weather was dry with a consistent traffic stream. Consequently, if the traffic composition changed suddenly producing a different capacity during the collection of a data sample, the increased variability would lower the acceptance of the data. Minor accidents along the freeway right-of-way, sharp variations in the intensity of rain during a peak period, or any unusual occurrence could produce fluctuations that the macroscopic flow model cannot adequately describe. Therefore, the test for acceptability was applied, and any nonuniform flow was discarded. The remaining samples could be taken to represent capacity flow under consistent comparable conditions.

ANALYSIS OF RESULTS

Results

The accepted capacity values, classified by subsystem and weather condition, are given in Table 4. It should be emphasized that these values are representative of an entire set of compatible data points, and that they survived a rigorous screening process. No value may be arbitrarily ignored as a freak, and each constitutes a very positive and definite representation of the capacity of the subsystem in question for the

TABLE 4
ACCEPTED CAPACITIES

Subsystem		Date	Capacity		
			Vehicles per Hour	Normalized	Dry/Wet
507	3	May 7	5,541	99.47	Dry
606	3	June 6	5,640	101.25	Dry
612	3	June 12	5,732	102.90	Dry
625	3	June 25	5,495	98.64	Dry
1021	3	Oct. 21	5,284	94.86	Dry
1031	3	Oct. 31	5,641	101.27	Dry
1105	3	Nov. 5	5,639	101.34	Dry
1106	3	Nov. 6	5,592	100.39	Dry
621	3	June 21	4,530	81.32	Wet
626	3	June 26	4,729	84.89	Wet
1212	3	Dec. 12	4,770	85.63	Wet
506	5	May 6	5,917	101.23	Dry
507	5	May 7	5,883	100.65	Dry
606	5	June 6	5,933	101.51	Dry
612	5	June 12	5,883	100.65	Dry
625	5	June 25	5,892	100.80	Dry
814	5	Aug. 14	5,710	97.69	Dry
1021	5	Oct. 21	5,689	97.33	Dry
1029	5	Oct. 29	5,853	100.14	Dry
621	5	June 21	4,685	80.15	Wet
1212	5	Dec. 12	4,995	85.42	Wet

conditions prevailing during the study period of approximately 2 hours during which the set of data was obtained.

In order to compare the dry weather capacities to wet weather, all the capacities were "normalized" for the 2 subsystems. This means that the subsystem 3 capacities were all reduced by the common factor necessary to result in a dry weather capacity mean of 100.00, and similarly for subsystem 5. It may be mentioned here that the dry weather capacity mean in subsystem 3 was 5,570.5, and in subsystem 5 it was 5,845.0. The subsystem 3 factor was therefore 100/5,570.5 and the subsystem 5 factor was 100/5,845. In this way all capacities could be compared to a dry weather mean of 100.00.

It can be seen that on the basis of a dry capacity of 100, the wet capacity is about 84. It is also obvious that the 16 dry weather capacities sampled all fall within a range of 94.9 to 102.9, or, a range of approximately 5 percent of the mean. Statistical methods were used on these data to establish the fact that rain has a highly significant effect on capacity, and that the wet weather capacity may be expected, with 95 percent confidence, to be between 81.2 and 85.8 percent of dry weather capacity. The dry weather capacities were all very closely bunched, so much so, in fact, that tolerance limits of 93.1 and 106.9 were calculated, within which 95 percent of the normalized dry weather capacities could be expected to lie. This provides some indication of the stability of capacity and its sensitivity to weather conditions. A range of ±7 percent of capacity is small, and it may therefore be concluded that capacity is sensitive to wet weather conditions.

Effect of Varying Acceptance Level

The question of how much these results would differ if the model acceptance level were changed may now be investigated. Table 5 gives the additional results of capacity and weather conditions that would have to be considered if the acceptance level were dropped from 10 to 1 percent. The capacity figures are normalized by application of the same multiplying factors for subsystems 3 and 5 that were used in Table 4.

It can immediately be seen that the inclusion of the capacities in Table 5 results in a bigger scatter of capacity values for each weather condition. An analysis of variance test on the entire set of results given in Tables 4 and 5 nevertheless show a highly significant difference between dry and wet weather capacities.

TABLE 5

CAPACITIES OF DOUBTFUL ACCEPTABILITY

Subsystem		Date	Capacity		
			Vehicles per Hour	Normalized	Dry/Wet
506	3	May 6	5,836	104.77	Dry
509	3	May 9	5,684	102.04	Dry
213	3	Feb. 13	4,795	86.08	Wet
923	3	Sept. 23	5,140	92.27	Wet
509	5	May 9	5,688	97.31	Dry
912	5	Sept. 12	5,762	98.56	Dry
508	5	May 8	5,792	99.09	Wet
626	5	June 26	4,733	80.98	Wet
923	5	Sept. 23	5,359	91.69	Wet

Note: Acceptance level between 1 and 10 percent.

TABLE 6

HOUSTON PRECIPITATION FREQUENCY OF OCCURRENCE

Rainfall (in.)	7-9 a.m.	4-6 p.m.	Both	
	Frequency per Year	Frequency per Year	Frequency per Year	Cumulative Frequency per Year
Trace	42.7	34.4	77.1	150.6
0-0.01	7.8	12.6	20.4	73.5
0.02-0.09	15.8	18.2	34.0	53.1
0.10-0.24	5.7	5.6	11.3	19.1
0.25-0.49	2.3	1.5	3.8	7.8
0.50-0.99	1.3	2.0	3.3	4.0
1.00-1.99	0.3	0.3	0.6	0.7
2.00 and over		0.1	0.1	0.1

Frequency of Occurrence of Rain

Because rain has been shown to have a significant effect in reducing freeway capacity, the chance that rain will occur during the peak period must be considered. It is doubtful whether any design or control planning should be modified for a capacity reduction that would rarely occur during the peak period. (Ample freeway capacity is assumed to exist in the off-peak hours.) General geographic location will be an important factor affecting frequency of rain. Most coastal areas will be subject to rain much more frequently than other inland areas. The Houston area weather records were examined to determine the frequency of peak-period rain.

From historical rainfall records, the number of times that a given amount of rain fell between specified hours in each month was calculated and accumulated for the entire year (5). Table 6 gives the average frequency for which various intensities of rain were observed during the hours from 7:00 a.m. to 9:00 a.m. and 4:00 p.m. to 6:00 p.m. in Houston. Most of the rainfalls recorded during the "wet" conditions for this research were on the order of 0.02 in. or more. Houston records indicate, then, that about 50 times per year the freeways will be operating under reduced capacity conditions due to rain. This would seem to be a high enough frequency to warrant consideration in design and control of the Houston freeways.

CONCLUSIONS

Based on the results of this research, the following conclusions can be made:

1. Rain significantly reduces freeway capacity.
2. The capacity of the freeway during rain can be expected to be between 81 and 86 percent of the dry weather capacity with 95 percent confidence.

3. Dry weather capacity is very stable as indicated by the fact that 95 percent of the dry weather capacity values could be expected to be within 7 percent of the mean observed capacity 99 percent of the time.

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Capacity Analysis of a Highway Link

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ABRIDGMENT

•THE PURPOSE of this work was to examine existing traffic conditions and to evaluate various improvements for increasing the capacity of a selected urban highway network. Two study methods were utilized: (a) a systems approach to capacity evaluation (utilizing stochastic methods where possible), and (b) data collection and analysis through the use of time-lapse photography.

The systems approach was used to formalize a realistic process for the evaluation of capacity on the study network when the overall impact of the environment on the network and the impact of the network on the environment were considered. The network was considered as a large subsystem of the system of major trafficways. It was then divided into 3 smaller subsystems, and analytical models were developed for each. The subsystems were (a) section 1, free-flow highway; (b) section 2, tollbooths located on a bridge; and (c) section 3, 2 major intersections providing the interface of the study with a freeway facility. Section 1 was analyzed by using procedures from the Highway Capacity Manual. Section 2 was analyzed by using queuing theory. The validity of the queuing theory models was tested by using the data collected from the time-lapse photography. Nomographs for the analysis of section 3 were developed and tested by using data from the time-lapse photography.

The time-lapse photography provided an inexpensive and rapid means for gathering extensive data. The technique proved very beneficial in testing the assumed probability distributions.

The study indicated that major reconstruction of the bridge, tollbooths, and intersections would be required to enable the network to handle the predicted volumes. Of major significance, however, was the fact that this study procedure indicated that such improvements would be of little value unless the freeway at the end of the network either was improved or had its demand volumes reduced.

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