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4 Reports

Subject Area

52 Road User Characteristics
55 Traffic Measurements

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Foreword

The subject of highway capacity continues to receive widespread interest among highway planners and designers, and therefore, by researchers also. The four papers in this RECORD are noteworthy additions to the literature on the subject.

In the first paper, Miller reviews progress on the Australian Road Capacity Guide. Of special interest in the work completed to date is the finding that capacity at signalized intersections is little affected by lane width variation from 10 feet to 13 feet. In addition, no correlation was found between saturation flow and city population.

Chang and Berry examined the intersection capacity charts in the 1965 Highway Capacity Manual for consistency, and found that certain individual curves are not entirely consistent with each other. Notes of caution are offered to users of the curves under particular conditions, and suggestions for improvement are made. In a discussion of the Chang and Berry paper by Miller, author of the first paper, some comparisons with the Australian guide are made.

Passenger car equivalency of trucks, at various percentages of truck volume, was investigated and reported by Reilly and Seifert in the next paper. Along with others, they questioned the Highway Capacity Manual statement that a truck on level tangent road is equivalent to two passenger cars, regardless of truck percentage. Their findings led them to conclude that under some conditions a truck appears to be equivalent to less than two cars.

In the final paper, Whitson, Buhr, Drew, and McCasland develop analysis techniques that can be applied to a freeway control environment. They conclude that it is now possible to provide systems to identify and react to freeway problems as they develop.

Contents

On the Australian Road Capacity Guide

ALAN J. MILLER, University of New South Wales, Australia

This paper reviews progress on the Australian Road Capacity Guide, the first part of which (on signalized intersections) was published during 1968. Some of the principal findings for signalized intersections were as follows. The capacity of an intersection approach is closely related to the number of lanes, and the width of these lanes has comparatively little effect over a range from approximately 10 to 13 feet. Capacities are calculated (in through-car units per hour) by adding those for each individual lane. On approaches with three or more lanes it was found that the utilization of the curb lane was usually poor, and formulas for estimating this usage have been derived in terms of the location of parked vehicles, the degree of enforcement of no-stopping bans, and the proportion of turning vehicles.

The paper lists the equivalents found for turning vehicles and trucks. For opposed right-turners (equivalent to American left-turners), the equivalent is a function of the amount of oncoming traffic and the signal settings. No correlation was found between capacity and city population.

The lost time per change of phase was found to be ½ second less than the inter-green time or 2½ seconds plus the travel time through the intersection of the last vehicle, whichever is greater. These conclusions were arrived at after collecting and analyzing data from 220 intersections in all Australian states.

New approximations have been obtained for the average delay, the average number of vehicles left in the queue at the end of the green, and the probability of clearing the queue, at fixed-time signals with random arrivals. A formula is also given for the load factor.

Overtaking studies have been carried out in both New South Wales and Victoria. These studies showed that a small proportion of drivers will not overtake, even when the vehicle in front is traveling at only 20 mph.

•THE Australian Road Research Board (ARRB) is sponsoring the Institute of Highway and Traffic Research (IHTR) to do the research necessary for the capacity guide. The original goal of the project was "to assemble or to produce traffic data for an Australian traffic capacity manual, and in doing so to utilize to the fullest the results of past studies and research by the various Australian state road authorities and of the School of Traffic Engineering (University of N.S.W.)." The project is controlled and guided by the Traffic Engineering Committee of the ARRB.

Work started in 1964 and is expected to continue for several more years. Expenditure to date has been about $120,000 (U.S.) and is expected to be about $50,000 (U.S.) for the current year. To speed up the project, the Institute has taken on additional staff, and part of the work is being undertaken by ARRB's staff.

The Director of IHTR, Prof. W. R. Blunden, is the project supervisor, and the senior staff members engaged full-time on the project at the IHTR are Dr. M. C. Dunne (formerly with General Motors in Detroit), Mr. R. J. Vaughan, and the author. At ARRB, Dr. R. L. Pretty and Mr. A. V. Forwood are working on the project.

Paper sponsored by Committee on Highway Capacity and presented at the 48th Annual Meeting.

PUBLICATION PLANS

The capacity guide is being published in sections as each part of the research is completed. An editorial committee, under the chairmanship of the author, has been appointed to prepare the guide. The first chapter, on signalized intersections, was published during 1968 as ARRB Bulletin No. 4 (1). When a number of the chapters have been produced, they will be revised as necessary and consolidated into one publication.

It is intended that all the data and assumptions on which the various statements, tables, formulas, and graphs in the guide are based be published. Because it would overburden the guide to include this material, it will be published as technical reports in the various ARRB publications as appropriate. Most of the data for the signalized intersection chapter are contained in ARRB Bulletin No. 3 (2). We feel it is important that the data and assumptions be published for the following reasons:

1. Users of the guide can see what degree of confidence they can have in it without having to wait to learn by experience.
2. In special cases not explicitly treated in the guide, the user can turn to these other publications for greater detail or for methods that may be helpful in solving his problem.
3. The data and methods are made readily available for others to further refine the work and show where further research is needed.

I would very much like to see published some of the evidence behind the U.S. Highway Capacity Manual, particularly that for the tables on the effects of lane width and lateral clearance (Tables 9.2, 10.2, and 10.8) and for the effect of bus stops at signalized intersections (Figures 6.11 to 6.14).

CAPACITY AND LEVELS OF SERVICE

In the provisional introduction to our capacity guide we have defined the capacity of a facility as the maximum sustainable rate at which vehicles can pass through it under a stated set of conditions, and added that the actual usage of a facility and its capacity are the same if the facility is fully saturated with traffic.

In measuring capacities, the shorter the period of observation the more variation there is in the observed rate of passage of vehicles. Very high flows can be observed by making the observation period short. What should be quoted as the capacity for practical use is the average flow of vehicles when a facility is fully saturated—or the best estimate possible from nearly saturated situations.

The task of estimating capacities from nearly saturated situations is one of the principal research problems in producing capacity data, and I believe it to be the main reason for most of the shortcomings with respect to the chapter on signalized intersections in the U.S. capacity manual.

Although it is useful to know the maximum amount of traffic that can use a facility, it is desirable that facilities rarely, if ever, be fully saturated. When the amount of traffic using a facility is close to capacity, speeds are slow, and there are long queues and delays.

In the first U.S. capacity manual the term "practical capacity" was introduced. This was a flow (not a capacity) somewhat less than the capacity of the facility and such that operating characteristics such as speeds and delays were subjectively considered to be just reasonable. This concept has been replaced with the "levels of service" concept, which is a qualitative way of describing operating characteristics.

In contrast, in Research on Road Traffic (3) the concept of an economic capacity is introduced. This is an attempt to define objectively a tolerable limit for the flow/capacity ratio. The economic capacity of a road is defined as "the total traffic passing along it in a year such that the most efficient improvements are just economically justifiable when the benefits to the road users are compared with the cost of the improvement." This definition leads to higher economic capacities in urban areas rather than rural because of higher land costs.

In the Australian guide, the approach will be to provide as much detail as possible on operating characteristics so that the user can decide how much excess

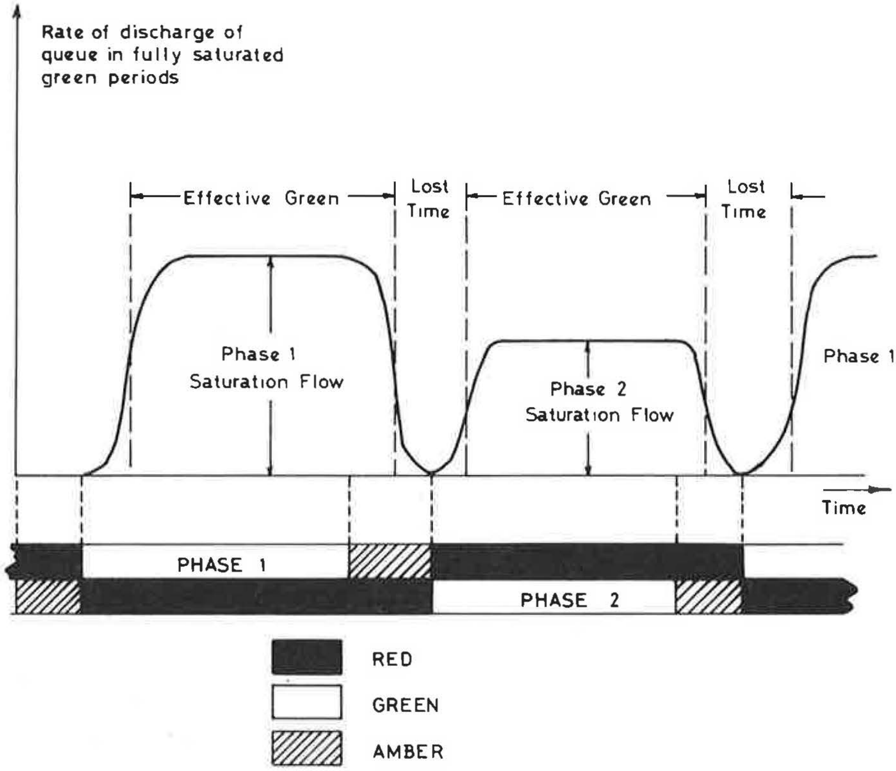

Figure 1. Basic model for signalized intersections.

capacity over the demand to provide and so that he can perform the economic evaluation of various projects.

SIGNALIZED INTERSECTIONS

Ross Blunden outlined our approach to the capacity of signalized intersections in his paper at the 47th Annual Meeting of the Highway Research Board (4). A few more of the details are given in the following.

Model

Until about 1960, all Australian state road authorities used the old U.S. capacity manual for signalized intersections. Webster's work (5) was published in 1958, and one by one the states changed to Webster's methods, mainly because of their simplicity but also because the results were found to be more reliable. By the time the new U.S. manual appeared, all states were using Webster's methods.

Figure 1 illustrates the Webster model. The saturation flow for an approach is defined as the rate at which vehicles cross the stop line after the first few, which are slow to start up, and until either the queue of stopped vehicles is exhausted or the phase ends, whichever comes first. The effective green time is usually almost the same as the actual green time.

The capacity of an approach depends on the fraction of the time that a green signal is showing for that approach, and is obtained by multiplying the saturation flow by g/c where g is the effective green time and c is the cycle time. If the arrival flow on one approach is q vehicles per unit time, then to provide adequate capacity on that approach we must have

$$s \frac{g}{c} \geq q \tag{1}$$

where s is the approach saturation flow. That is, the green time fraction must be such that

$$\frac{g}{c} \geq \frac{q}{s} \tag{2}$$

The capacity of any approach can be increased by allocating to it a larger fraction of the available green time, but this is at the expense of the other approaches. The condition for the capacity to be adequate to meet the demand on all approaches is that the inequality (Eq. 2) is satisfied for all approaches. Now since the cycle time, c, is the sum of the phase lengths plus the lost time L (usually about 5 seconds per change of phase), we can obtain the following inequality by summing both sides of Eq. 2 for each phase:

$$1 - \frac{L}{c} = \sum_{i=1}^{n} \frac{g_i}{c} \geq \sum_{i=1}^{n} \frac{q_i}{s_i} \tag{3}$$

where the suffix i is the number of the phase, and n is the number of phases. Where two or more approaches or movements share the same phase, the one with the higher value of q_i/s_i should be used. The ratios of arrival flow to saturation flow have become known as y-values from Webster's notation, so the inequality (Eq. 3) means that the sum of y-values must be less than 1 - L/c. If this condition is satisfied, then it is possible to find phasings to provide adequate capacity on every approach. Notice that the quantity 1 - L/c can be increased slightly by increasing the cycle length but is reduced by increasing the number of phases.

Capacity calculations for signalized intersections thus reduce to two stages: (a) estimate saturation flows, and (b) calculate the sum of y-values. If the sum of y-values is less than some predetermined value (we recommend 0.7), then there is adequate capacity and a reasonable amount to spare. If not, then ways to increase the capacity need to be found. The value 0.7 is derived from studying operating characteristics, and more will be said about this subject later.

Notice that the calculations do not involve either the phase length or cycle length. This is particularly valuable in Australia, where almost all signals are either fully vehicle-actuated or part of linked systems (sometimes vehicle-actuated). Only one state still has fixed-time signals at isolated intersections; most states have never installed such a signal.

One of the basic assumptions of the Webster model (also implicit in the U.S. capacity manual) is that the saturation flow is independent of phase lengths. This is not the case in two situations: (a) for lanes that do not continue on the exit side of the intersection or that are blocked by parked vehicles on either side of the intersection, and (b) for lanes containing opposed turning vehicles without a separate phase—right-turners in Australia, left-turners in the United States. Models have been developed for each of these cases. These models both depend on the phase lengths and involve iterative processes, but often sensibly guessed values of g and c give adequate accuracy.

Saturation Flows and Lost Times

In his paper for the Highway Research Board's 47th Annual Meeting (4), Blunden pointed out that we are making one fundamental departure from both British and American methods by relating saturation flows to the number and type of lanes rather than to approach width. This stems from some excellent work by H. J. W. Leong of the N.S.W. Department of Main Roads (6). He measured saturation flows at a number of intersections in Sydney and found that for practical purposes lane width has a negligible effect on saturation flows in the range from 8 to 11½ feet.

The British decision to relate saturation flows to approach width rather than to the number of lanes was made after experiments disclosed an almost linear relationship between saturation flow and approach width with no evidence of "steps". However, these experiments were conducted without lane lines striped on the approaches.

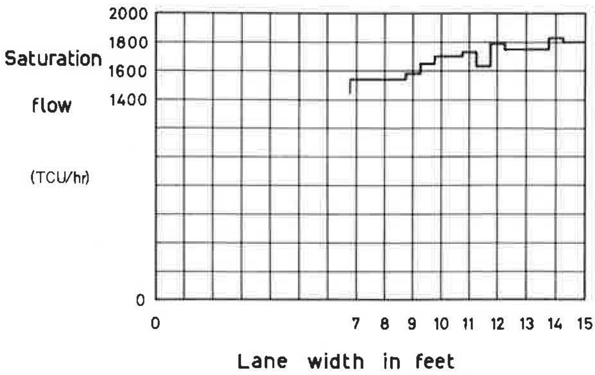

Figure 2. Saturation flow vs lane width after adjustment for location, gradient, and lane type.

Wide streets and control of parked vehicles encourage good lane discipline. In Britain many streets are very narrow, with the most common approach width probably about 15 to 16 feet. With a moderate proportion of bicyclists and motorcyclists in the traffic stream as well, it seems reasonable to relate saturation flows to approach width for most intersections in Britain.

In Australia, streets are generally much wider. A common street width is one chain (66 feet) from curb to curb. I could find only six sets of signals, including two at pedestrian crossings, with single-lane approaches in the whole of Australia. There are more intersections with six lanes on one approach.

Figure 2 shows the relationship found between saturation flow and lane width in the data collection for the Australian capacity guide. This is based on just over 600 lanes at about 220 intersections, but after a number of adjustments for location, gradient, etc. First I will discuss the data collection and the model, then return to this figure.

A team of three from IHTR visited each state and, with the help of between 6 and 12 locally recruited staff, collected data at a stratified random sample of intersections in the state. Two methods of measurement were used. At a small number of intersections, tapeswitch detectors and Esterline-Angus pen-recorders were used to obtain

TABLE 1

PRINCIPAL FACTORS AFFECTING SATURATION FLOW AND METHODS OF ESTIMATION

Factor	Possible Methods of Estimation			
	Within Sites	Between Sites	Experimentally	Theoretically
Geometrics				
Width of approach		X	X	
Number of lanes		X	X	
Width of lanes		X	X	
Gradient		X		
Radius of turns		X	X	
Operating Conditions				
One-way or two-way		X		
Parking on approach		X	X	X
Parking on exit		X	X	
Weather	X	X		
Bus stops	X	X		X
Environment				
Metropolitan population		X		
Intersection environment		X		
Traffic Characteristics				
Turning movements	X	X	X	X
Trucks	X			
Trams	X	X		X
Bicycles and motorcycles	X	X		

$$\log_e(s) = a + b_i + c_j + d \cdot w + e_k + f \cdot g + \varepsilon$$

in TCU/hour per lane · city · location · lane width · lane type · gradient · residual

Testing of model

1. First order interactions investigated by adding cross-product terms and testing for a significant reduction in the residual sum of squares. Only one interaction, location x lane type, was statistically significant (and very highly). An extra factor was added for left turn lanes in the C.B.D.
2. Effect of lane width examined. Effect found to be non-linear. Two-gradient model substituted with bend at 10'.
3. Effect of weather studied. 3 classes - rain, overcast and fine. No significant differences between classes.

Figure 3. Saturation flow model.

headway measurements accurate to about 0.1 second. A light-sensitive resistance was used so that signal changes could also be accurately recorded on the charts. This method was used for determining through-car equivalents and lost times. At the majority of intersections, observers using tape recorders called out vehicle types and turning movements as vehicles crossed the stop-line. A stopwatch was used to time phases and to find the time taken for the queue to clear. This method is adequate for most practical purposes. Full details of the two methods are given elswhere (2).

Many factors influence saturation flows. Some of these are shown in Table 1, together with the ways in which the effects of these factors may be inferred. As a first stage in our analysis we looked at those effects that can be estimated from data for a single intersection. These effects are those checked under "Within Sites" in Table 1. From the accurate headway measurements we found the following:

1. Two consecutive trucks have less effect than two separate ones. The difference was highly significant statistically but unimportant practically.
2. Allowing for the effect of a truck on the next lane as well as its own, on the average a truck is equivalent to 1.85 through cars.
3. Two consecutive left-turning (U.S. right-turning) cars have the same effect as two separate ones, and each is equivalent to 1.25 through cars.
4. A left-turning truck is equivalent to 2.4 through cars.
5. A formula was derived giving a vehicle equivalent for opposed right-turning cars and trucks in terms of the signal phases and the opposing flow. The average equivalents found were 2.9 for cars and 3.9 for trucks.

From this stage, all saturation flows were converted to through-car units (TCU's). In our method, capacities are expressed in TCU's and are then independent of the percentages of turning vehicles and trucks. To allow for these percentages we scale up the demand to TCU's per hour, rather than the U.S. method of scaling down the capacity. The two methods are exactly equivalent, although it is difficult to calculate capacities in vehicles per hour when summing lane saturation flows. The difficulty is that it is necessary to know the distribution of turning vehicles and trucks between lanes.

Figure 3 shows the model we used to study the variation in lane saturation flows between different intersections. The figure explains the testing of this model. The lane types referred to were

L—lanes containing left-turners (not necessarily 100 percent), plus right-turn lanes from one-way streets,
T—lanes containing only through vehicles, and
R—lanes containing right-turners (not necessarily 100 percent), excluding right-turn lanes from one-way streets.

It was necessary to transfer the right-turners from one-way streets from type R to type L because these turns are strongly affected by pedestrians and because the radius of curvature is usually very short.

The effects of all factors in Figure 3 were found to be statistically significant at the 5 percent level. No evidence of any relationship between saturation flow and city population was found, although there were significant differences among cities. The range of city populations was from 56,000 to 2,500,000.

In Figure 2 we saw that the effect of lane width is very small over the range we were able to consider—from $6\frac{1}{2}$ to about 16 feet. Probably a greater effect would be noticeable if the average widths of lanes on each approach had been used, rather than the widths of particular lanes in plotting this figure; the $6\frac{1}{2}$-foot lane was on an approach with two other lanes of $9\frac{1}{2}$ and 10 feet. No special study of trucks in narrow lanes was made, although an analysis of average headways alongside trucks showed no correlation with lane width.

Another important factor that has to be considered when using the lane-by-lane approach is that the curb-lane capacity is often not fully utilized. Figure 4 shows our findings in this regard.

1. Without parking

   100 % usage on 1 or 2-lane approaches.

   40 % } usage on approaches with 3 or more lanes.
   60 %

   no stopping strictly enforced

   Usage > 40 % (or 60 %) if
   i) there is a heavy left-turn movement (U.S. - right turn),
   ii) there are freeway conditions downstream.

2. Parking downstream - within 600'.

   100 % usage on 1 or 2-lane approaches.
   Only $1\frac{1}{2}$ through vehicles per phase on approaches with 3 or more lanes.

3. Parking on approach, but not exit.

   Approximately 1 vehicle per 30' goes into the curb lane during the red. After this initial queue in the curb lane has cleared, the saturation flow drops to that for a single through lane at the point A.

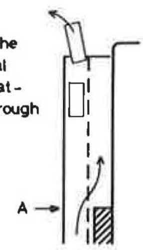

Figure 4. Curb-lane usage.

Figure 5 shows the overall result for saturation flow against approach width for a suburban shopping location with level approaches. For comparison, the almost straight line is that given by the British Road Research Laboratory (7). It has been assumed that there are no parked vehicles. It is difficult to make such a comparison with the U.S. manual. The result depends on whether one-way or two-way streets are chosen and what city population is used. A rough comparison shows that the Australian and U.S. figures are roughly the same for 3- and 4-lane approaches provided the lanes are at least 12 feet wide and the population is over 1,000,000. For other numbers of lanes, the Australian capacities are higher, except in the CBD where they are slightly lower.

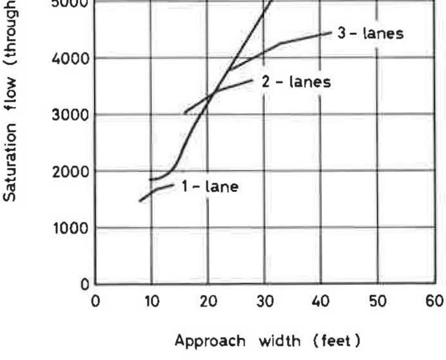

Figure 5. Saturation flow vs approach width using Australian and British formulas.

The lost time per change of phase is approximately $\frac{1}{2}$ second less than the inter-green time provided that this is sufficiently long for the last vehicles going through on one phase not to delay the start of traffic in the next phase.

In a saturated phase, the average time at which the last vehicle crossed the stop line was about 3 seconds after the end of the green. In a small number of cases it was 5 or more seconds after the end of the green. It usually takes between 1 and 3 seconds for the vehicle to travel through the intersection area so that the inter-green time should ideally be between 5 and 7 seconds after a saturated phase.

We recommend that the lost time per change of phase be taken as either (a) the inter-green time minus $\frac{1}{2}$ second, or (b) $2\frac{1}{2}$ seconds plus the travel time through the intersection of the last vehicle, whichever is longer.

Operating Characteristics of Signalized Intersections

There appear to be no published formulas for the operating characteristics of vehicle-actuated signals except for the case in which there are no maximum or minimum phase lengths. In heavy traffic or when the vehicle extension has been set too long, vehicle-actuated signals operate almost as fixed-time signals.

For fixed-time signals there is an abundance of formulas for operating characteristics (2, 5, 8, 9, 10, 11, 12, 13). Most of these formulas are for random arrivals and a constant number of vehicles able to pass through the intersection each phase, although at least two of them allow for non-random arrivals and variations in the possible numbers of departures. We believe that most of these formulas are sufficiently accurate for practical use. In the case of the well-known Webster formula (5), there have been many verification studies in different parts of the world. For simplicity and reasonable accuracy, we chose to recommend the following formulas and provided nomograms to simplify calculating values:

$$\bar{d} = \frac{c - g}{2c(1 - y)} \left[\frac{2}{q} E(z) + (c - g)\right] \tag{4}$$

$$E(z) = \frac{\exp(-1.30\ \phi)}{2(1 - x)} \tag{5}$$

$$\phi = \frac{1 - x}{x} \sqrt{sg} \tag{6}$$

$$p_0 = 1 - \exp(-1.58\ \phi) \tag{7}$$

where $\bar{d}$ is the average delay per vehicle on one approach, $E(z)$ is the average number of vehicles left in the queue when the signal changes back to red, and p_0 is the proportion of phases in which the queue is cleared. The other symbols are as in the usual Webster notation:

g = effective green time;
q = arrival flow;
s = saturation flow;
c = cycle time; and
x = degree of saturation $= \frac{qc}{sg}$.

These formulas do not hold if $x \geq 1$, i.e., if the capacity is exceeded. In using the formulas, the units must be consistent. For instance, if times are in seconds, flows must be in vehicles per second.

The load factor (LF) used in the U.S. capacity manual is approximately $1 - p_0$; the difference is the small probability that a phase is loaded, but the signal changes just as the last vehicle is crossing the stop line so that no vehicles are left behind. Using the simulation results of May and Pratt (14), a good approximate formula for the load factor is

$$LF = \exp(-1.3\ \phi) \tag{8}$$

By considering these operating characteristics for 2-, 3-, and 4-phase intersections and with high and low saturation flows, a recommendation that the sum of y-values should not exceed 0.7 was derived. It was suggested that 0.75 be used as the absolute limit.

Peak-hour factors are not used. We recommend that for design purposes the peak half-hour be used except in small cities, where the peak 15-minute count should be used. We suggest the use of 5-minute counts in these small cities to locate the peak 15 minutes.

RURAL ROADS

Work on the rural roads part of the project has only just begun although some measurements of speeds and some overtaking studies have been carried out. As in the case of signalized intersections, we hope to be able to produce reliable formulas for calculating both capacities and operating characteristics. One of the applications for which quantitative, rather than qualitative, estimates of operating characteristics are required is in cost-benefit studies.

Plans for this part of the project have been outlined elsewhere (15), and are as follows:

Stage 1—from empirical studies, to develop relationships between free speeds and geometric features, roadside development, weather, etc.;

Stage 2—to develop appropriate queuing models so that actual speeds, delays, and queue sizes can be related to free speeds and the flow of traffic in each direction; and

Stage 3—to relate the parameters of these queuing models (e.g., following headways, gap-acceptance in overtaking) to geometrics, opposing flow, weather, etc.

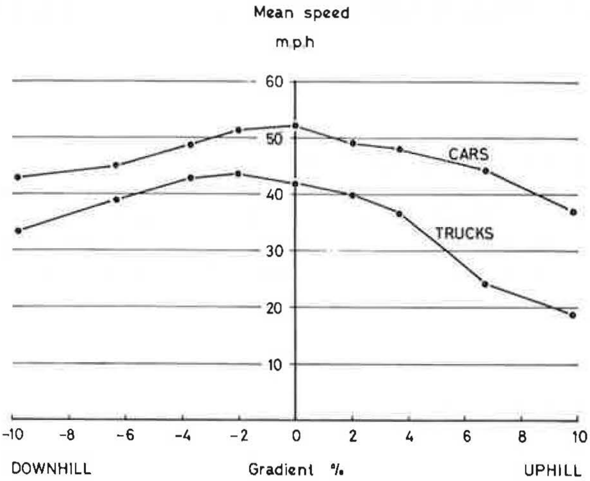

Figure 6. Mean speeds of cars and trucks vs gradient (after adjusting to 2000-ft sight distances) on rural two-lane roads in N.S.W.

Stage 1—Free Speeds Related to Geometrics

Two independent analyses of free speeds (15, 16) have shown that the average coefficient of variation (i.e., the ratio of the standard deviation to the mean speed) on Australian rural roads is about 17 percent. This is close to results found in Britain (17).

Leong's paper (16) at the recent ARRB conference contained some useful results on the effect of geometric features on free speeds. Figure 6 is based on Leong's data and shows mean speeds of cars and trucks against gradient after applying linear sight distance corrections.

Stage 2—Development of Queuing Models

A number of queuing models will be necessary to cover 2-lane roads, 4-lane roads, 2-lane roads with an added climbing lane, 4-lane roads plus climbing lane, etc. The most important of these for Australian conditions are the 2-lane roads.

The basic components of any queuing model for rural roads are the rates and patterns of catching up and overtaking. If queue lengths, speeds, and delays can first be related to these, then the two rates can later be empirically related to geometrics, gap-acceptance in overtaking, etc.

The major difference between queuing in rural traffic and standard queuing theory is that the number of queues is not constant. Each vehicle other than the slowest can catch up to slower vehicles, and each vehicle other than the fastest can be caught by faster vehicles. This means that we cannot clearly segregate vehicles into the customers and servers of standard queuing theory.

The rate at which vehicles catch up to one another was probably first derived by O. K. Normann (18), although it is one of those results that seems to be capable of everlasting rediscovery. Later Wardrop (19) showed how Normann's tedious calculations could be largely eliminated. When vehicles can overtake freely, he showed that the rate at which overtakings occur is, per unit distance, per unit time, approximately

$$0.56\ \sigma\ k^2 \tag{9}$$

where σ is the standard deviation of vehicle speeds and k is the concentration of vehicles per unit distance. If the coefficient of variation of speeds is 18 percent, the rate of overtaking in terms of the flow (volume) q is, per unit distance, per unit time,

$$0.1 \frac{q^2}{\bar{v}} \tag{10}$$

where $\bar{v}$ is the space-mean speed.

Simple models have been proposed (20, 21) for the cases where there is an equilibrium between catching up and overtaking, and for the case in which overtaking is impossible. In these models it was found that average queue lengths were almost linearly related to the following dimensionless quantities (z and y). For the equilibrium case,

$$z = \frac{q}{\lambda(1 - x)} \tag{11}$$

and when overtaking is impossible

$$y = \frac{qX}{\bar{v}(1 - x)} \tag{12}$$

where λ is the average rate in overtakings per unit time at which vehicles are able to pass a slow vehicle while there is a queue behind it, x is the ratio of flow to capacity, X is the distance for which overtaking is impossible, and $\bar{v}$ is the space-mean speed of all vehicles.

The models referred to are both very elementary, but they do give an understanding of the interaction of many variables that will greatly help in the analysis of empirical data, or in fitting a sensible function to simulation results.

Some preliminary work has been done on a model that will incorporate sight distances. The first simulation runs were designed to test whether or not the distribution of straight, level sections suitable for overtaking makes any appreciable difference to delays. A 6-mile length of road was simulated with overtaking possible on 3 miles. The 3 miles were divided into one, two, and four equal lengths in different runs. It was found that it is much better to have several short sections on which overtaking is possible than one long section.

Moskowitz and Newman (22) have developed a model for queuing on hills when there are two lanes for the uphill direction. Their results are approximate, although the approximation is good for light to moderate traffic. It is hoped that their results can be improved to hold for all traffic flows and to use the "no-overtaking" work (21) for hills with only one lane for the uphill traffic.

Stage 3—Measurement of Queuing Model Parameters

Preliminary studies of gap acceptance in overtaking have been made in both New South Wales and Victoria. So far the studies have concentrated on the acceptance or

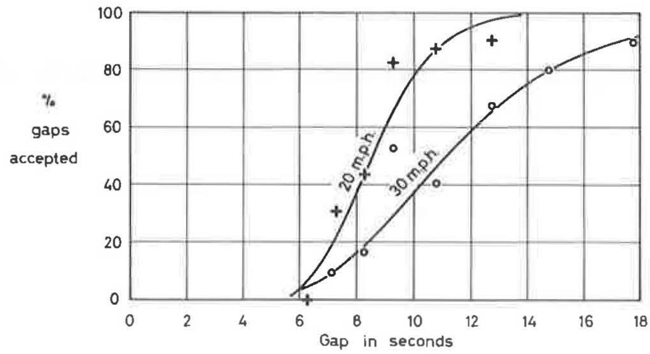

Figure 7. Percent gaps accepted vs size of gap for drivers overtaking 20-mph and 30-mph vehicles.

rejection of gaps in the opposing stream by vehicles that have slowed down to the speed of the vehicle to be overtaken. Only a small amount of data on the effect of sight distance has been collected.

An important factor is the speed of the vehicle to be overtaken. On a straight two-lane road it is much easier to overtake a 30-mph vehicle than one traveling at 40 mph. To estimate the effect of speed, vehicles were driven at constant speed with observers inside them noting when other vehicles caught up and overtook and when opposing vehicles passed.

Figure 7 shows the proportions of gaps of various sizes that were accepted by the drivers of queuing vehicles in the N.S.W. studies. The quantity plotted is quite different from the proportion of drivers who will accept a given size of gap. The latter proportion is always higher. Notice that we have chosen to measure gaps in time, not distance. The size of gap was the interval between opposing vehicles passing the slow car.

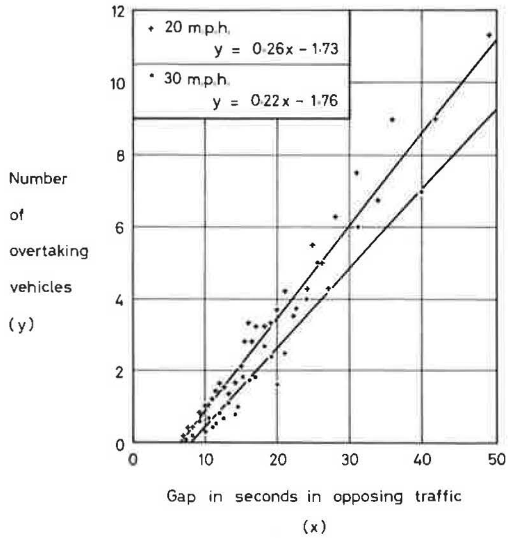

Figure 8. Number of vehicles that overtake a slow vehicle vs the gap in the opposing traffic.

At both 20 and 30 mph, the sample size was just under 200 gaps of 7 or more seconds. The fitted curves were obtained from a log-probit analysis. The observations were all made on weekends on the same test section. The sizes of gaps that were accepted in 50 and 90 percent of the cases were $8\frac{1}{2}$ and 11 seconds respectively at 20 mph and 11 and 17 seconds at 30 mph.

Figure 8 shows the number of vehicles that overtook vs the size of gap when there was a long enough queue behind the observation car for the gap to be fully utilized. This type of analysis is more useful than the gap acceptance analysis in studying the capacity for overtaking.

In the studies in Victoria, higher speeds were used (30, 35, and 40 mph), and it was possible to have 8 to 10 observation cars for some of the studies. At these speeds much shorter queues formed behind the observers' cars, and a few slower vehicles were caught up. It was not possible to obtain a figure similar to Figure 8 because queues were not long enough.

For each driver who caught up to an observation car and slowed down, the largest gap he rejected and the gap accepted, if any, were recorded. Assuming a log-normal distribution of critical gaps between drivers, the parameters of the gap-acceptance distribution were estimated using the method of maximum likelihood. During the studies it became apparent that a small percentage of drivers will not overtake in any circumstances while the slow vehicle is still moving, and we had to incorporate into the model a proportion p of such drivers. Table 2 gives the critical gaps that 50 percent of drivers would accept and the estimated proportions of non-overtakers.

TABLE 2

GAPS ACCEPTED BY 50 PERCENT OF DRIVERS

Speed of Slow Vehicle (mph)	Time Period	Accepted Gap Length (sec)	Proportion p of Non-Overtakers (percent)
30	Weekend	10.3	1
35	Weekend	11.4	4
35	Weekday	11.5	$1\frac{1}{2}$
40	Weekend	12.3	7

Notice that the time gap increases with the speed of the vehicle to be overtaken. This is in contrast with the results of the only similar study of which we are aware (23). Notice also that the time gap is only weakly dependent on the speed of the vehicle to be overtaken. Crawford found the 50 percent critical gap to be about 11 seconds (23).

Full details of these overtaking studies are given by Ashton et al (24) and Miller and Pretty (25).

UNSIGNALIZED INTERSECTIONS

Work has only just begun on the unsignalized intersections stage of the project. The results will be unique to Australia since, as far as I know, it is the only country in the world with the "give-way-to-the-right" priority rule and with driving on the left-hand side of the road. Except for a few streets in Tasmania and Canberra, there are no "major" and "minor" streets. A stop sign in Australia means just what it says; it does not also mean give-way as in many countries.

The methods and results of this part of the project will have little or no value outside of Australia, so no further details will be given here.

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Examination of Consistency in Signalized Intersection Capacity Charts of the Highway Capacity Manual

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Intersection capacity charts of the 1965 Highway Capacity Manual have been analyzed for consistency, giving major emphasis to actual vs expected differences in results when changes are made in parking conditions and in width of approach for one-way and two-way operation. Comparisons are also made with methods for computing capacity as developed in Great Britain and Australia. Findings indicate that the individual curves in the 1965 Manual are not entirely consistent with each other. Consistency would be improved by shifting two of the curves to different positions (two-way streets with parking and one-way streets with parking on both sides). Care should be taken when using the present curves to determine effects of such changes as prohibiting parking, changing from one-way to two-way operation, or changing the lane markings on wide approaches.

•CAPACITY CHARTS of the 1965 Highway Capacity Manual are based primarily on empirical data from 1,100 heavily traveled intersection approaches in the United States (1). This paper analyzes the curves shown on these charts from the standpoint of consistency between expected differences in capacity for changes in width of approach, parking vs no-parking conditions, and one-way vs two-way conditions. Expected differences are based in part on logical reasoning. Comparisons are also made with theory and results of capacity studies from England and Australia.

Consistency among the different charts is important, since the Manual is frequently used to estimate possible changes in capacity and levels of service that may result from such changes as widening, prohibiting parking, prohibiting turns, or establishing one-way streets. Unless the different charts are consistent, they will not provide realistic information on possible effects of such changes.

BACKGROUND

Analyses of the consistency of intersection capacity charts published in 1950 (2) and in 1958 (3) were made by Wattleworth (4). He analyzed capacity in terms of expected additional capacity as additional width is provided by widening or by prohibiting parking.

Figure 1 compares the 1965 capacity curve with 1950 and 1958 curves for two-way streets with no parking. In general, the shape of the 1950 curve is concave downward, and the approaches are less efficient for each additional increment of width. The major criticism of the 1950 curves was that the capacity values were too low in downtown areas, especially for two-way streets with parking. Across the country, many streets had been observed to carry higher volumes than had been considered possible by the charts. Traffic engineers in the Chicago area applied a factor of 1.4 to the capacity

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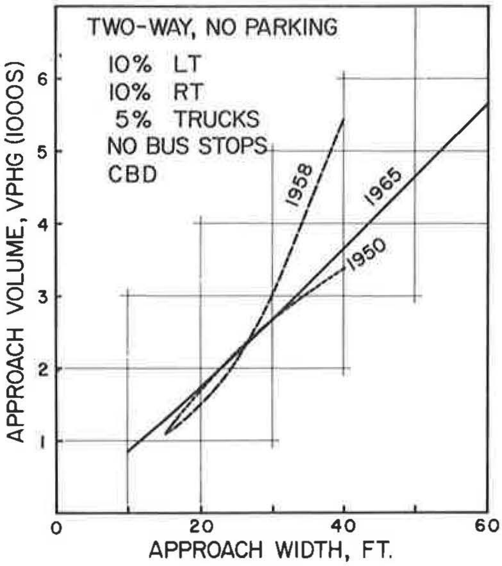

Figure 1. Capacity curve of 1965 Manual compared with possible capacity curves of 1950 and 1958.

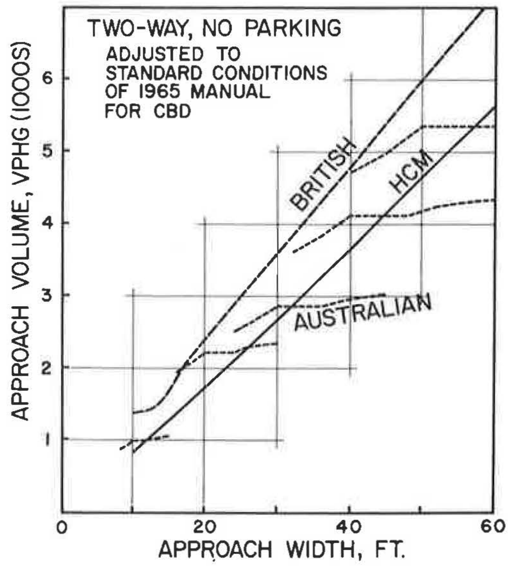

Figure 2. Comparison of intersection capacity curve of the 1965 Manual with British and Australian values.

derived from the 1950 curves. The Chicago Area Transportation Study (5) also had found inconsistencies in terms of the effect of street width and type of area on capacities.

The general shape of the 1958 curve is concave upward. Wattleworth (4) pointed out that for wide approaches, the capacity volume was too high. According to his calculations, the slope of the curve for a wide two-way street exceeds the maximum possible slope.

The 1965 Manual curve is concave upward also, but is closer to a straight line. For wide approaches it is a straight line, which means that the capacity is directly proportional to the width of the approach. The 1965 curve generally lies between the extremes of the 1950 and 1958 curves, in terms of both the shape and the capacity values.

Figure 2 compares the Manual curve with the British and Australian capacity curves (6, 7) for two-way streets with no parking, 10 percent left turns, 10 percent right turns, and 5 percent trucks and through buses. The British and Australian curves were corrected for these percentages, using average values of 1.75 and 2.90 equivalent through-car units (TCU's) for right and left turns respectively. The British curve is a straight line except for the range in width of 10 to 17 feet, and yields higher capacity values than the other two curves. Moyer and Casey's discussion of Leong's paper (8) pointed out that the average length of car might have some effect on capacity at low speeds, and reported average passenger car lengths of 13 feet in the United Kingdom, 15 feet in Australia, and 17.5 in the United States. It is possible that the narrower average width of cars in Britain also permits more lanes on wider streets.

The Australian intersection capacity curve is a step function. It provides for higher capacity values when a wide approach is marked for a greater number of lanes. The average values of each curve are similar to those of the Manual curves except for the two-lane approach. The effect of lane width is indifferent for widths of 10 to 12 feet; the capacity then increases as lane widths exceed 12 feet and decreases as lane width decreases below 10 feet.

The British and Australian methods are based on a combination of theory and field study. In both methods, a theoretical approach was first used to obtain the effect of opposing traffic on right turns for two-way streets (left turns in the United States).

Miller (7, 9) presented a formula for the through-car equivalents for right turns, E_{rt}, as follows:

$$E_{rt} = \frac{1.5}{\left[f \frac{sg - qc}{g(s - q)} + \frac{4.5}{g} \right]} \tag{1}$$

where q and s are values of the flow and the saturation flow for the opposing traffic in TCU's per hour, g and c are the green time and cycle time in seconds, respectively, and f is a function of the opposing flow (varying from 1.0 at low opposing flows to 0.45 at values of q = 800).

Another theoretical approach by Miller (7, 9) is based on the probability, p_o, that no vehicles are left in the queue, expressed as

$$p_o = 1 - \exp\left(-1.58\ \sqrt{sg}\ \frac{1 - x}{x}\right) \tag{2}$$

where x is the degree of saturation, qc/sg, and s is in TCU/sec. This relationship is shown in Figure 3. The probability of the queue being exhausted is related not only to the degree of saturation but also to the saturation flow. Load factor in the Highway Capacity Manual would generally correspond to $1 - p_o$.

DEVELOPMENT OF THEORETICAL CURVES

The development of theoretical curves for U.S. conditions is hampered by lack of adequate research data on effects on capacity of changes in parking, changes in approach widths, percent of turns and turn controls, and changes to one-way operation. However, the general shape and properties of the curves can be identified from logical reasoning and consideration of previous research.

Basic Model

The basic condition first examined was a signalized approach on a street with no parking, no turning vehicles, and no commercial vehicles.

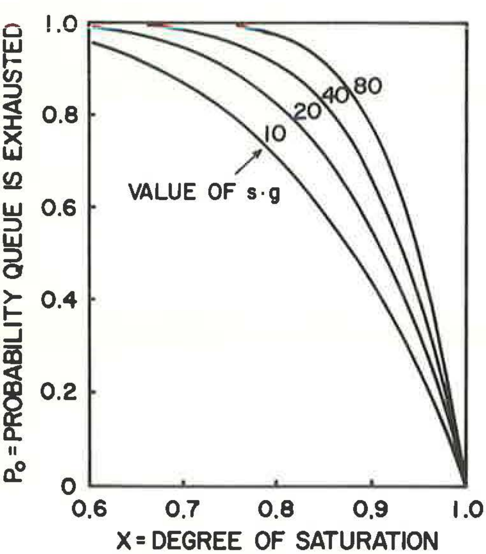

Figure 3. Probability of exhausting a queue vs degree of saturation (7).

The minimum width of a lane is related to the size and operational characteristics of vehicles. A one-lane approach with a minimum lane width of W feet will discharge a saturation flow of s vehicles per hour of green. If a multiple-lane approach consists of lanes of equal width, W, the saturation flow of the approach will be integral multiples of s and will be proportional to the number of lanes.

The saturation flow of a lane should increase as its width increases because of easier maneuverability of vehicles, more comfortable feeling on the part of drivers, etc. However, there should be a maximum effective width of a lane beyond which the saturation flow of the lane does not increase. This maximum effective width of a lane is designated as W + W′, which has a saturation flow of s + s′. If it is assumed that the saturation flow is linearly proportional to the width of the lane, then the slope of the saturation flow line is s′/W′ and the resulting basic model is as shown in Figure 4.

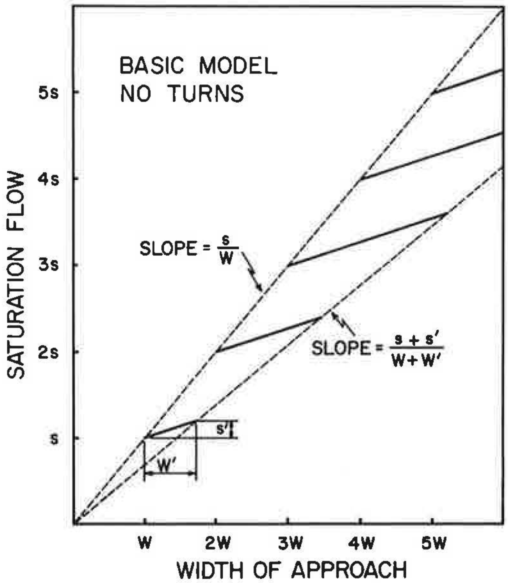

Figure 4. Basic model of saturation flow vs width of intersection approach.

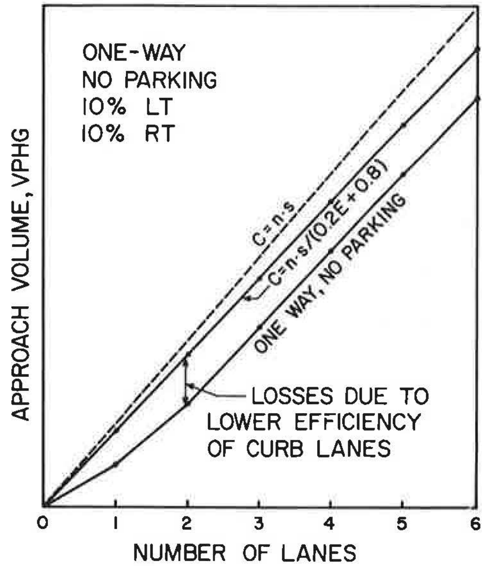

Figure 5. Expected relationships between capacity and number of lanes on one-way streets as affected by turns and use of curb lanes.

If s' is large enough for s'/W' to be equal to s/W, the saturation flow line becomes a single straight line that passes through the origin with a slope of s/W, as shown in Figure 4. The Manual curve corresponds to this type of intersection capacity model (Fig. 2).

For approach widths of 10 to 15 feet, the British curve (Fig. 2) corresponds more nearly to the Australian curve, with s'/W' less than s/W. This conforms more nearly to the theoretical curves in Figure 4 for this range of approach widths.

No attempt is made here to develop a theoretical saturation flow curve for each lane. Such a curve might be a straight line—in this case the slope should be less than s/W—or it might be a series of curves like the Australian curves shown in Figure 2. Instead, an attempt is made to further refine the general shape and properties of the line that connects the saturation flows for an entire approach composed of integral multiples of minimum width lanes (in the model, the straight line which passes through the origin with slope of s/W) by consideration of factors affecting capacity.

For the convenience of examining the consistency in the Manual's charts, standard conditions are considered to be 10 percent left turns, 10 percent right turns, and no local transit buses or trucks.

Capacity Curve for One-Way Streets With No Parking

For one-way streets, the effects of right and left turns can be considered the same, with the through-car equivalent of each turning vehicle equal to E, where $E > 1$. Then the capacity for an n-lane approach with 10 percent left turns and 10 percent right turns is

$$C = \frac{ns}{(0.2E + 0.8)} \qquad (3)$$

where s is the saturation flow of a lane. Hence, as long as E does not vary with street width, the capacity curve is a straight line, shown as the upper solid line in Figure 5. The exact location of this line will depend on whether E varies with street width and the extent to which pedestrians interfere with turning movements.

According to Australian research (7, 9), an additional loss in capacity results from reductions in carrying capacity of curb lanes. The lowest of the three curves in Figure 5 takes into account the probable loss in capacity resulting from normal curb lanes being considered undesirable because of vehicles standing or stopping to load and unload passengers. The exact position of this curve will vary, depending on the extent to which curb lanes on the right and left sides are used for loading, unloading, or standing. The slopes of the two solid lines should be the same for widths of more than two lanes.

Capacity Curve for Two-Way Streets With No Parking

The major additional factor to consider in characterizing the capacity curve for two-way streets is the effect of opposing traffic on left turns. The limiting condition occurs when the opposing flow is at capacity and the intersection is controlled by two-phase signals. Under these conditions, left turns can be made only during the change of the phase from green to yellow. With such a limiting condition, most two-way approaches of two or more lanes cannot accommodate 10 percent left turns. Thus, the volume of opposing traffic must be sufficiently low so that the through-car equivalent for left-turning vehicles, as computed by Eq. 1, will not be so high that left-turn capacity controls the capacity of the approach.

The capacity for an n-lane approach with 10 percent left turns and 10 percent right turns can be expressed as

$$C = \frac{ns}{0.1\,(E_l + E_r) + 0.8} \tag{4}$$

where s is saturation flow for a lane, and E_l and E_r are through-car equivalents for a left and a right turn, respectively.

The value of E_r can be considered to be constant for different widths of streets (unless there is substantial pedestrian interference or lack of exit capacity). However, the value of E_l may vary with approach width. Furthermore, more left turns must be made for wide approaches than for narrow ones, assuming left turns equal 10 percent of total approach volume.

Miller (9) presents a table of E_{rt} values (corresponding to E_l in the United States), in which E_{rt} increases as street width increases. We applied these to two-way street approaches of increasing widths—as indicated by increasing saturation flows—taking into account that for 10 percent left turns, the number of left turns increases for the wider approaches. The Australian results indicate increasing effects of left turns as saturation flows increase (shown by changes in computed left-turn reduction factors in Table1). In contrast, the Manual indicates that the effects of 10 percent left turns decrease as street widths increase (Table 1). This difference warrants further study.

From these analyses we can conclude that capacity for a two-way approach with left turns is less than that for one-way streets because $E_l > E_r$. The other effects (pedestrians, curb lane usage, etc.) should be the same as with one-way streets. Therefore, the theoretical two-way curve could be derived by substracting from the one-way curve the additional effects of opposing traffic on left turns. The resulting curve is shown in Figure 6.

Capacity Curves for Streets With Parking

Parked vehicles reduce the width available for moving traffic, and passing drivers tend to shy away from the parked vehicles. This loss of effective approach width is assumed as constant, W″, for all approaches. Loss of capacity due to parking turnover can be taken as C′ for each line of parking. Thus, if the capacity curve for streets with no parking is expressed as

$$C = f\,(W) \tag{5}$$

TABLE 1

EFFECT OF SATURATION FLOW AND OPPOSING FLOW ON CAPACITY REDUCTION FACTORS FOR 10 PERCENT LEFT TURNS ON TWO-WAY STREETS WITH NO PARKING

No. of Lanes	Saturation Flow for Entering and Opposing Directions, vphg	Opposing Flow, q, vphg	E_l[a]	Approach Volume at Load Factor = 1, vphg		Approach Reduction Factors for 10 percent Left Turns	
				Total	Left Turns	Australia Data	United States Data
For Low Opposing Flow							
1	1,800	200	1.7	1,682	168	0.93	0.77
2	3,600	400	2.1	3,243	324	0.90	0.91
3	5,400	600	2.4	4,736	474	0.88	0.95
For Higher Opposing Flow							
1	1,800	400	2.3	1,593	159	0.89	0.77
2	3,600	800	3.1	2,970	297	0.83	0.91
3	5,400	1,200	4.1[b]	4,125	413	0.76	0.95[c]

[a]Based on E_{rt} values from Appendix B, Australian Road Capacity Guide, for C = 60 and g/c = 0.6.
[b]From correspondence with Dr. Alan Miller.
[c]Left-turn capacity is exceeded, because q + LT > 1200.

where W is the width of an approach, then the capacity curve for streets with parking is

$$C_p = f\ (W - W'') - C' \tag{6}$$

The capacity curve for one-way streets with parking on both sides is

$$C_p = f\ (W - 2W'') - 2C' \tag{7}$$

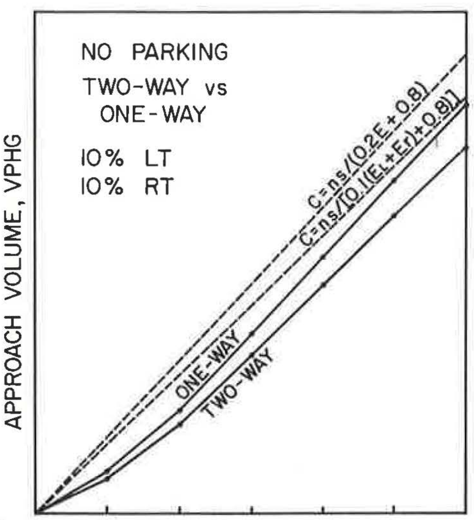

Figure 6. Expected relationships between capacity and number of lanes for two-way vs one-way streets.

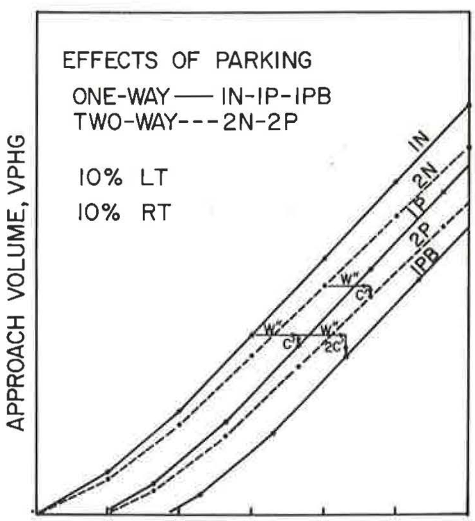

Figure 7. Expected relationships between capacity and number of lanes as affected by parking.

The resulting curves for streets with parking are shown in Figure 7.

Service Volume Curves for Various Load Factors

Because the load factor is the proportion of the number of loaded green intervals to total green intervals during the peak hour, it follows that flow is reduced below capacity when the load factor is below 1.0. Saturation flow is the flow at a load factor of 1.0. At first glance the ratio of the reduced flow to the saturation flow is likely to be the same for all approach widths. But Figure 3 and Eq. 2 indicate that (a) for a fixed p_o, the degree of saturation increases with increases in the saturation flow, and (b) the flow curve for load factors below 1.0 may be concave upward rather than a straight line. Other research (10, 11) indicates that further study is needed to identify shapes and relative positions of the curves for load factors below 1.0.

EXAMINATION OF CONSISTENCY

Examination of consistency of the charts in the Manual is based on theoretical considerations developed in the preceding section. The first relationship examined is whether capacity should be related mainly to the width of approach regardless of the number of lanes of the approach. According to the Manual charts, a 60-foot approach would yield the same capacity if it is striped for either four, five, or six lanes. This is questionable, as has been pointed out in the previous section. However, the Manual does indicate in its Table 6.2 the optimum number of lanes for different ranges of width.

Capacity Curves

Five Manual curves showing capacity values for approaches in central business districts (CBD) are superimposed in Figure 8. Curve 1N, for one-way streets with no parking, passes through the origin if extrapolated and is concave upward up to a 50-foot approach width, which corresponds to four to five lanes, and then becomes a straight line. In this respect, the curve fits well with the theoretical curve shown in Figure 5.

Curve 2N, for two-way streets with no parking, also passes through the origin if extrapolated and is concave upward to a two- to three-lane approach width, followed by a

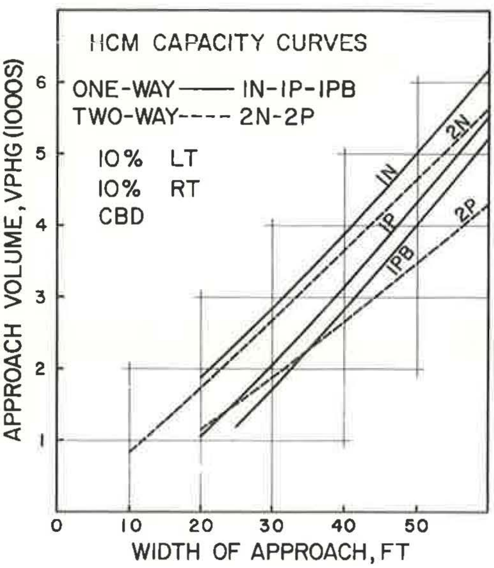

Figure 8. Manual curves for one-way and two-way streets as affected by parking.

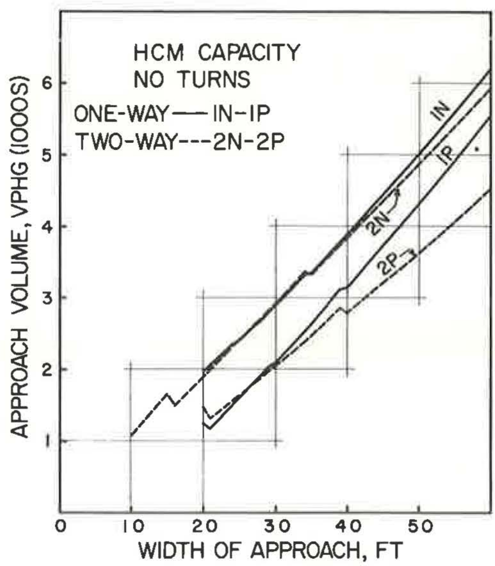

Figure 9. Manual curves for one-way and two-way streets, adjusted to no left turns.

straight line. This curve can be considered a fair fit when compared with the theoretical curve of Figure 6.

Curve 1P, for one-way streets with parking on one side, intersects with the width-axis at a point about 10 feet from the origin if extrapolated as might be expected. It is concave upward, followed by a straight line, conforming generally in shape with theory. Because the curve for one-way streets with parking on both sides is similar to that for one-side parking, it also agrees with the theoretical curve as to shape. However, the position of curve 2P, for two-way streets with parking, does not correspond with theory. It should have intersected, if extrapolated, with the width-axis at the same point as curve 1P.

One-Way vs Two-Way

Theoretically, if no left turns are involved, there should be no differences between one-way and two-way street capacities for the same approach widths.[1] Figure 9 compares the curves for one-way and two-way streets, adjusted for no left turns according to Tables 6.4 and 6.5 in the Manual. For streets with no parking (1N and 2N), the difference is not great. However, the capacity for one-way streets with parking on one side is as much as 22.5 percent greater than the capacity of two-way street approaches of similar widths with parking. This amounts to a 1,020 vphg difference for a 60-foot approach.

Figure 8 can also be used to compare the one-way and two-way curves, with 10 percent left turns, for streets with no parking and for streets with parking on one side of each approach. The difference between curves 1N and 2N for streets with no parking seems to be in accordance with expected differences as discussed earlier. However, for approaches with parking on one side, curves 1P and 2P intersect at a width of 24 feet. For approaches less than 24 feet wide, two-way capacity is greater than one-way capacity. For consistency, it would seem that these two "parking-on-one-side" curves in Figure 8 should bear the same relation to each other as for the one- and two-way curves (1N and 2N) for streets with no parking.

Another problem relates to the curves showing the capacity of wide two-way streets with 10 percent left turns. As mentioned earlier, wide approaches cannot accommodate 10 percent left turns if the opposing volume is heavy and there is a two-phase signal. If we compute the capacity of a 60-foot approach to a two-way street with no parking and with a separate left-turn lane of adequate length and assume 10 percent left turns, 10 percent right turns, and 5 percent trucks, the total opposing approach volume cannot be greater than 723 vphg for 5,650 vphg to be discharged from the studied approach. This represents an extreme unbalance in flow by direction. Apparently, both capacity and service-volume curves for wide two-way streets with 10 percent left turns are valid only if highly unbalanced flows are expected, or if the signal phasing provides for protected left turns. Perhaps the basic curves should be shown for zero percent turns.

Parking vs No Parking

Essentially there should be no differences, in terms of the shape of the curves, between Manual curves for streets with parking and with no parking. The curves should coincide by a parallel shifting, either vertically or horizontally. The Manual curves shown in Figure 8 are in general conformity with this principle.

Differences in capacity between the curves for parking on one side of a one-way street (curve 1P) and no parking (curve 1N) vary from 830 vphg at 30 feet to 700 vphg at 50 feet of approach width, as shown in Figure 8. Removing parking from a street that formerly allowed parking on one side is equivalent to adding from 8 to 6 feet of street width, according to these Manual curves. However, the differences in going from

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[1]For streets with no left turns, the capacity of one-way streets could be less than for a two-way approach of the same width, because the one-way street has two curb lanes. However, the left curb lane is less likely to be blocked by loading and unloading than is the right curb lane.

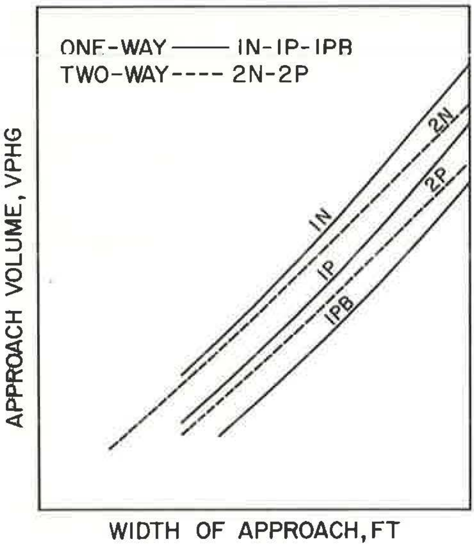

Figure 10. Positioning of manual curves to improve consistency for average conditions for trucks and turns in CBD.

parking on both sides (1PB) to parking on one side (1P) yields values of capacity additions of less than 350 vphg. Logically, the capacity additions by changing from parking on two sides to parking on one side should be the same as in changing from parking on one side to no parking on either side of a one-way street. Apparently, one of the three curves for one-way streets should be shifted if the three one-way curves are to be consistent with each other.

The differences in capacity between the curves for parking (2P) and no parking (2N) on two-way approaches in Figure 8 should also be similar to the differences in eliminating parking on one side of a one-way street, as shown in Figure 7. However, the added capacity from prohibiting parking on two-way streets is greater than expected, increasing as much as 1,100 vphg green on a 50-foot approach. One or both of these curves apparently needs repositioning.

Fringe Areas

Discrepancy in capacity values between one-way and two-way streets is even greater when fringe-area adjustment factors are applied to the curves shown in Figure 8. The curve for two-way streets with parking is shifted upward due to the 1.25 adjustment factor, whereas the curves for one-way streets with parking remain as shown in Figure 8.

Improving Consistency

Consistency of the Manual capacity curves would be improved if curve 2P (two-way streets with parking) were positioned just below curve 1P (one-way streets with parking), as is shown by Figure 10. Curve 1PB (one-way streets with parking on both sides) would be more consistent with others if it were approximately 700 vphg below curve 1P. The exact position of each curve cannot be specified from this preliminary study.

Further study should be given to analyzing relative positions of these curves, taking into account the original date and the consistency factors discussed in this paper.

Level of Service Curves

An attempt was made to examine consistency of the Manual curves for load factors less than 1.0 (12). These involve studies of bulk queuing and studies of probability of clearing queues. May and Pratt (10) have also studied these differences by computer simulation. Studies in these areas are continuing.

CONCLUSIONS

1. The examination of consistency of the intersection capacity curve of the 1965 Highway Capacity Manual has revealed some inconsistencies that could produce misleading results when using the Manual to determine possible effects of prohibiting parking or changing from two-way to one-way streets.

2. The relationships between the Manual's intersection capacity curves for different widths of one-way vs two-way conditions and parking vs no parking should be examined carefully, and adjustment made to improve consistency in capacity values. Specific points indicated from this study for CBD conditions include the following: (a) curves derived for two-way streets with parking and for one-way streets with parking on one side should be identical for zero percent left turns, whereas they differ considerably;

similar curves for the no-parking condition on one- and two-way streets are practically identical, as expected; (b) the capacity curve for one-way streets with parking on both sides appears to yield values too high, as compared with other one-way street curves; and (c) effects of parking do not seem to be consistent on one- and two-way streets; the Manual curve for two-way streets with parking appears to require adjustment in slope, as shown by the dashed line in Figure 10.

3. Adjustment factors for fringe-area conditions appear to need revision.
4. Further research on effects on capacity and level of service for changes in widths, lane markings, parking, turns, and opposing flows is desirable to quantify values that can be used as consistency restraints in preparing the next editions of the signalized intersection chapter of the Highway Capacity Manual.
5. The Australian approach of analyzing intersection capacity by studying characteristics of flow by lane seems to have advantages.

ACKNOWLEDGMENT

The assistance of Dr. A. J. Miller of the University of New South Wales is acknowledged in comparing the British, Australian, and U.S. Manual approaches to capacity and level of service, and in planning this study.

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Discussion

ALAN J. MILLER, Institute of Highway and Traffic Research, University of N.S.W., Australia—I approach this paper with considerable trepidation. I am terrified that Messrs. Chang and Berry may apply the same treatment to our Australian capacity guide!

This paper points to the importance of having a logical and systematic model for intersection capacities. Not only does this avoid most of the dangers of inconsistencies, but it aids comprehension and enables the user of the Manual to make better judgments in the case of complex, irregular intersections.

Chapter six, on at-grade intersections, is possibly the weakest part of the Highway Capacity Manual, based as it is mainly on data collected in 1955 and 1956. I doubt if there has been much increase in capacities since then, but there has been a considerable refinement in models for intersection capacity, and the operating characteristics of signalized intersections are much better understood now. It is hardly surprising that there are inconsistencies in the Highway Capacity Manual. The relevant question is, "How important are they?"

The answer to this question depends on the use of the Manual and the accuracy of prediction required from it. For a new intersection or in a computer assignment of traffic in a network, the predictions of traffic demand are often so crude that a very rough estimate of capacity is adequate.

The situation requiring accuracy is in the modification of an existing intersection to squeeze just a little more capacity from it. Here I do not think users can have much confidence in the Highway Capacity Manual. It tells us that we cannot increase the capacity by striping more lanes in the same approach width; it does not tell us the effect of banning parking for various distances on the approach and exit sides of intersections; and Chang and Berry have questioned whether it tells us correctly the effect of turning vehicles. In addition, our experience in Australia has suggested that the effects of local buses (Figures 6.11 to 6.14 of the Manual) are too large except in the CBD. Since the striping of lane, control of parking, banning or phasing of turns, and location of bus stops are four of the principal tools of the local traffic engineer, I feel that Chapter 6 of the Highway Capacity Manual largely fails to meet his needs.

Lest I appear to be advertising our Aussie guide, let me add that it is mute on the effect of bus stops, and on a few other factors that I will not mention, in case Messrs. Chang and Berry turn their attentions "down under".

Professor Berry sent me a copy of Table 1 in advance of the rest of the paper. I had not attempted to convert our "through-car equivalent" method into the "reduction factors" of the Highway Capacity Manual and I was interested to see the results. At first one might expect that the effect of left-turners would be greatest for single-lane approaches where one left-turner stops everything, whereas on a three-lane approach, two of the lanes continue moving. The explanation of why Table 1 shows the opposite is in two parts. First, it is the percentage of left-turners that is fixed, not the number. This means that the single lane is moving for most of the time, whereas on a three-lane approach, one lane will consist almost entirely of left-turners. The second reason is that the amount of opposing traffic has been increased in proportion to the number of lanes. This means that, after the queue of opposing vehicles has cleared, the left-turners on a single-lane approach will have a good chance of finding a gap, whereas on the three-lane approach, left-turners will often have to wait until the change of phase before they can make their turns.

Y. B. CHANG and DONALD S. BERRY, Closure—We appreciate the comments of Dr. Miller, as well as the assistance he provided earlier in the planning of this study.

Truck Equivalency

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

A two-lane, dual-dual, 50 mph, level, tangent roadway having a 1967 AADT of 68,000 vehicles with a high percentage of trucks was studied under uninterrupted flow conditions. A relationship between mixed volume (cars and trucks on a local roadway) and equivalent passenger car volume (only cars on an express roadway) was determined for 20, 40, 60 and 80 percent truck groups. The passenger car equivalent of trucks was found to approach a value of two as the percent of trucks in the stream approached 100 percent. The relationship between cars and trucks was determined on the basis of equal speeds.

•THE Highway Capacity Manual states that a truck on a level, tangent road is equivalent to two passenger cars, regardless of truck percentage. Other studies show that this value may not be as high as two, and may vary with percentage of trucks. Whatever its value, the relationship can be shown in the form of a mixed volume-equivalent passenger car volume graph as the truck percentage varies. To derive this relationship, volumes and speeds on a local roadway for various truck percentage groups were compared to volumes and speeds on an express roadway.

The following definitions of terms have been used throughout this report:

Truck—a vehicle with more than four tires;

Dual-dual highway—a highway with four separated roadways: an express roadway and a local roadway for both directions;

Express roadway—a two-lane roadway with only passenger car traffic;

Local roadway—a two-lane roadway with both passenger car and truck traffic;

Equivalent passenger car volume—an expanded volume (vph) from the express roadway that includes cars only;

Mixed volume—an expanded volume (vph) from the local roadway that includes both cars and trucks;

Factor—the passenger car equivalent of a truck as determined from the relation $Q = C + F(T)$, where Q is equivalent passenger cars, C is the number of cars, T is the number of trucks, and F is a factor;

Variable lane headway—the difference in time (seconds) between consecutive vehicles (regardless of lane) crossing a reference line; this term applies to vehicles in adjacent lanes as well as vehicles in the same lane (vehicles are measured head to head; if two vehicles are crossing the reference line at the same time, the variable lane headway is zero).

The dual-dual section of US 1-9 in Elizabeth, New Jersey, was an ideal study site. The roadway was level and tangent, flow was uninterrupted, and a high percentage of trucks was included in the 1967 AADT of 68,000 (both directions). Based on a 16-hour count in 1958, there were approximately 25 percent trucks over the entire roadway, or approximately 50 percent trucks on the local roadway, northbound. This road has a posted speed limit of 50 mph.

STUDY METHOD

Using a pneumatic-tube, junior counter, 20-pen recorder system, the following data were collected: chronological times at which a vehicle crossed a pair of reference lines, vehicle type (car or truck), and lane used (right or left). Three 12-hour days

Paper sponsored by Committee on Highway Capacity.

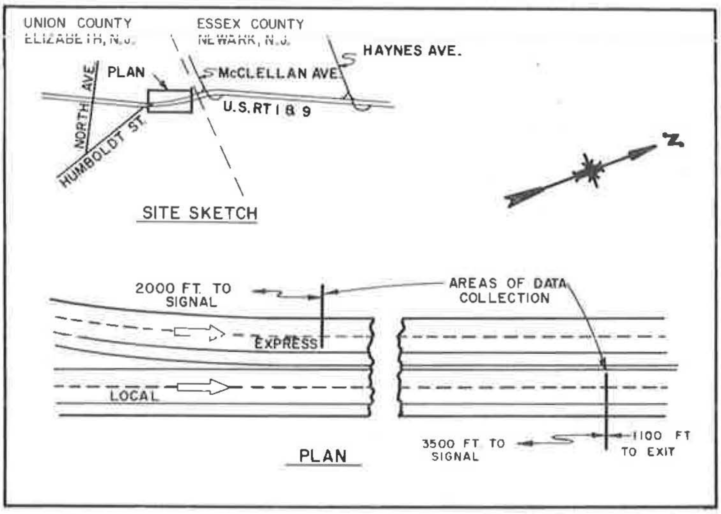

Figure 1. Site sketch and plan.

of data from the express roadway and two 12-hour days of data from the local roadway were collected. Although data were collected by individual lanes, both lanes were used in computing volumes and average speeds.

The data were coded, punched onto cards, and compiled by computer programs. The first programs (AVGSPD, PLATON, and PLASUM) defined the data using time interval and platoon methods. Tables of average speed, with volume and truck percentage, were produced. From these tables, second-degree curves of volume vs speed were fitted, using the computer program polynomial curve fitting. These curves were extended to 35 mph and adjusted to a maximum volume of 4,000 vph for a two-lane road. Then 20, 40, and 60 percent truck curves were constructed equidistantly between the express curve and the 80 percent truck curve. These curves were used to plot a mixed volume-equivalent passenger car volume relationship. This relationship is based on the assumption that, for a constant speed, the mixed volume is equivalent to the corresponding express roadway volume.

DATA COLLECTION

Site

The study site was US 1-9 in Elizabeth, New Jersey, a dual-dual highway, downstream from a signalized intersection (Fig. 1). The northbound direction, which includes a local and an express roadway of two lanes each, was studied. The speed limit was 50 mph.

The express roadway site was 2,000 ft downstream from the traffic signal, and the local roadway site was 3,500 ft downstream from the traffic signal. There is no access within 1,000 ft of the local roadway site.

The local roadway is tangent, with a 10-ft shoulder on the right and an 18-in. wide tapered concrete separator to the left. The express roadway has a 1-deg curve to the left, a 9-ft shoulder on the right, and a sloping curb and a wide grass median on the left. Both roadways have 12-ft wide portland cement concrete lanes and bituminous concrete shoulders.

Express roadway data were taken during 1967 on Friday, May 26, from 8:30 a.m. to 7:00 p.m.; Wednesday, May 31, from 7:15 a.m. to 7:00 p.m.; and Thursday, June 1, from 7:15 a.m. to 7:15 p.m. Local roadway data were taken on Wednesday, June 14, from 7:30 a.m. to 7:15 p.m.; and Thursday, June 15, from 6:30 a.m. to 7:15 p.m.

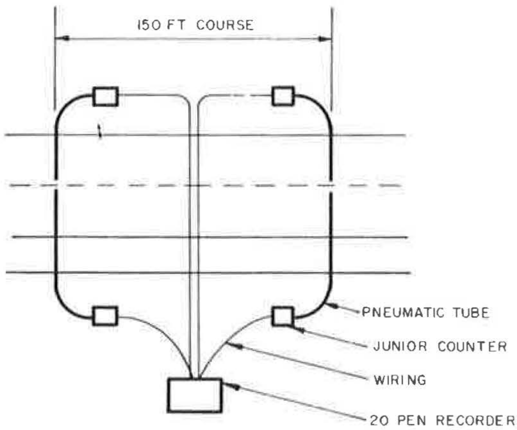

Figure 2. Data collection system.

The weather was clear and the roadway was dry during the data collection periods.

Collection Method

As described earlier, a pneumatic-tube, junior counter, 20-pen recorder system was used to collect the data (Fig. 2). Two reference lines were established 150 ft apart. Pneumatic tubes along these lines recorded each lane separately. The time at which a vehicle crossed a tube was registered, and vehicle type (car or truck) was manually classified.

DATA COMPILATION

The recorder chart consisted of six lines; each lane had a line for truck indication and a line for each end of the speed course. To find the time difference for a vehicle on the 150-ft course, "blips" on two adjacent lines had to be matched. Speed, in 5-mph intervals, was determined by measuring this time difference with a specially constructed scale. Chronological time at which a vehicle crossed a reference line, vehicle type, lane, and speed were coded for each vehicle.

Time Interval Method

A constant time interval was used to define data samples. All vehicles (regardless of lane in the time interval), starting with the first vehicle, made up the first sample. Thereafter, time samples were taken consecutively to the end of the study.

The first set of samples was formed using 15-second time intervals, and another set of samples was formed using 30-second time intervals. The number of vehicles in a time interval was expanded to vehicles per hour.

If there were less than three vehicles in a sample or if more than half the speeds were missing for the vehicles in a sample, then that sample was not used.

Platoon Method

Samples were also defined by platoons. A platoon was defined using variable lane headway criteria between consecutive vehicles (in time), regardless of lane. If a pair of consecutive vehicles were in the same lane, the second vehicle was assumed to be influenced by the first vehicle for variable lane headways up to 6 seconds. If a pair of consecutive vehicles were in adjacent lanes, the second vehicle was assumed to be influenced by the first vehicle for variable lane headways up to 3 seconds. In the event that two vehicles passed the reference line at the same time in adjacent lanes, the 6-second criterion was used for a following vehicle.

A condition could occur where the variable lane headway between two adjacent lane vehicles (vehicles 4 and 5 in Fig. 3) is greater than 3 seconds, but the headway between two same-lane vehicles (vehicles 4 and 6) is less than 6 seconds. For this condition, neither vehicle 5 nor 6 is considered to be influenced by vehicle 4. Vehicle 6, in this position, is assumed to be primarily influenced by vehicle 5. Vehicles 1, 2, 3, and 4 form a platoon. Vehicles 5 and 6 are the start of another possible platoon. A valid platoon was required to have a minimum of four vehicles and a minimum total platoon time (i.e., variable lane headway sum) of 15 seconds. Volume was

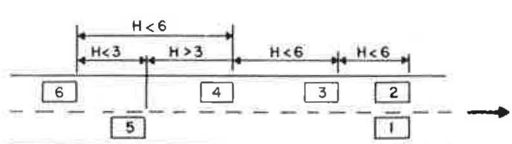

Figure 3. Special condition for platoon method.

determined by expanding number of vehicles/total platoon time to an hour. Volumes were grouped by 100-vph class intervals.

Procedure

For each sample of data, the expanded hourly volume, average speed, and truck percentage were determined. Each sample was then grouped by volume and truck percentage. Trucks were grouped as follows:

Roadway	Percent of Trucks	Assumed Percent of Trucks
Express	0.0	0.0
Local	0.0- 5.0	0.0
	5.1- 29.9	20.0
	30.0- 49.9	40.0
	50.0- 69.0	60.0
	70.0- 89.0	80.0
	90.0-100.0	95.0

For each volume and truck percentage group, the average speed, the standard error, and the number of samples were tabulated (see Tables A-1, A-2, and A-3 in Appendix).

DATA ANALYSIS

A second-degree relationship between volume and speed was determined for each truck percentage group and for each method (15-second time interval, 30-second time interval, and platoon). At this point some of the curves were eliminated—the 15-second curves due to wide variations and the 95 percent truck curves due to small sample size and low volume range. The data have a speed range between 40 and 50 mph, but the curves were extended to 35 mph.

The maximum volume (4,000 vph for a two-lane road, according to the Highway Capacity Manual) was assumed to occur at 35 mph. Since the volume at 35 mph for both methods exceeded 4,000 vph, the curves were corrected by a factor of 4,000 vph/volume at 35 mph. The corrected corresponding truck percentage curves for the 30-second time interval and platoon curves were then averaged together. The 60 percent truck group curve was an exception; only the platoon curve was used in this case because the 30-second curve was so flat (Table 1 and Fig. 4).

TABLE 1

SPEED-VOLUME CURVES[a]

$$Y = AX^2 + BX + C$$

Y = Speed (mph)

X = Volume (vph)

Coefficient	Express	Truck Percent 0	20	40	60[b]	80
A	-1.11×10^{-6}	-3.26×10^{-6}	-1.62×10^{-7}	-1.25×10^{-6}	-1.60×10^{-6}	-5.42×10^{-6}
B	1.78×10^{-3}	6.49×10^{-3}	3.31×10^{-4}	7.41×10^{-4}	1.42×10^{-3}	1.16×10^{-2}
C	4.57×10^{1}	4.46×10^{1}	4.45×10^{1}	4.60×10^{1}	4.40×10^{1}	3.75×10^{1}

[a]These curves represent the average between the 30-second time interval and the platoon methods, adjusted so that the express volume is 4,000 vph at 35 mph.

[b]The 60 percent curve is based only on the platoon method.

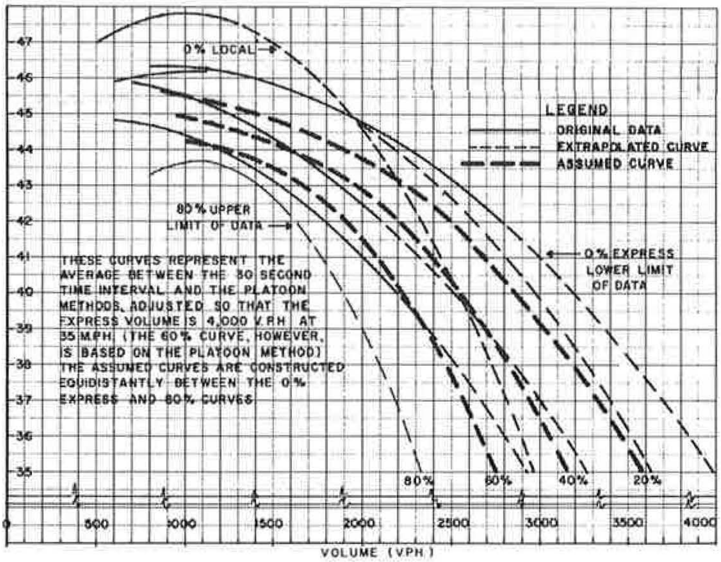

Figure 4. Speed-volume curves for varying truck percentage groups.

Many speed-volume curves showed a speed increase with volume to a peak speed at the lower volumes, and then the expected speed decrease. Only the decreasing portions of these curves were used.

It was found that the speed-volume curves had an almost uniform variation between the limiting express and 80 percent truck curves. Therefore, assumed curves were constructed for 20, 40, and 60 percent truck groups. (A family of curves was thus formed that consisted of the limiting curves and the assumed curves.) In the speed range from 35.0 to 43.7 mph, the assumed curves were constructed equidistantly between the limiting curves. For speeds greater than 43.7 mph, the assumed curves were extended to proportioned points along a chord between the vertices of the two limiting curves (Fig. 4 and Appendix Table A-5). This family of curves was used to plot a mixed volume-equivalent passenger car volume graph.

Mixed Volume-Equivalent Passenger Car Volume Graph

Using the family of speed-volume curves in Figure 4, equivalent passenger car volumes can be determined for each truck percentage group using the express curve as the base. The following is an example, using 40 percent trucks:

Speed (mph)	Volume (vph)	
	Mixed	Express
35	3,200	4,000
40	2,500	3,200

Because the express volume consists of all passenger cars, it is considered to be the equivalent passenger car volume at the given speeds. Hence, Figure 5 is constructed using express volume as the ordinate and mixed volume as the abscissa.

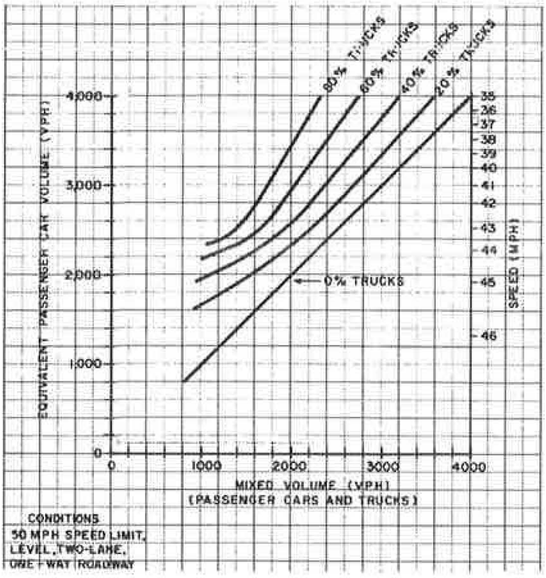

Figure 5. Mixed volume vs equivalent passenger car volume.

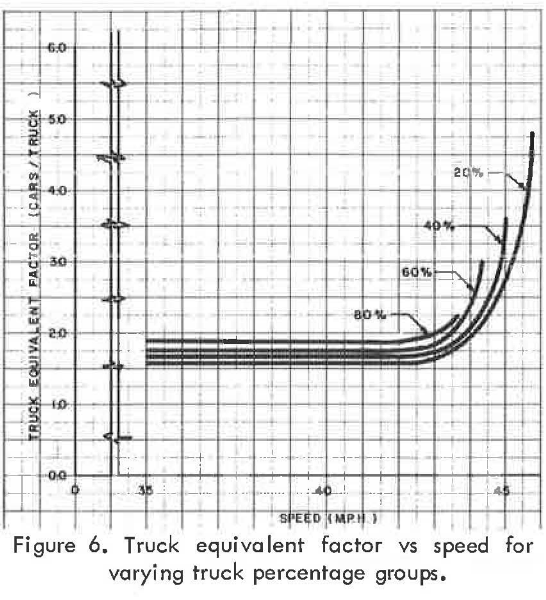

Figure 6. Truck equivalent factor vs speed for varying truck percentage groups.

Truck Equivalent Factor Curves

A factor (defined earlier) was computed for each truck percentage group for speeds between 35 and 46 mph. These factors are tabulated in Appendix Table A-6 and are shown in Figure 6.

DISCUSSION OF RESULTS

Speed-Volume Curves

The expected pattern for the various truck percentage groups is a family of decreasing curves with the express curve outermost (Fig. 4). Some exceptions occur in the 15-second curves, with both the 60 and 80 percent truck group curves flatter than expected. The 30-second, 60 percent truck group curve was inexplicably flat. The zero percent local truck curve was expected to approximate the express curve but did not. It was initially above and then fell below the express curve (Fig. 4). No explanation could be found for the higher speeds on the local roadway at the lower volumes. However, the local curve data cover a shorter and lower volume range than the express curve data, and an extension of this curve cannot be expected to yield much similarity to the express curve. This may explain the difference at the higher volumes where the local roadway data were extended.

In comparing the same truck group curves (by percentages), the time interval curves fall below the platoon curves. This can be expected because of the different techniques used to compute volume. Platoon volume is based on the average headway from the first to the last vehicle in a platoon. The time interval is used as the base for volume in the other method. Because there is "lost" time in the time interval method, the computed volume for the time interval method will appear to be slightly less than for the platoon method.

These curves were extended to 35 mph, based on the assumption that the maximum capacity for all truck percentage groups occurs at this speed. However, maximum capacity for high truck percentage groups could possibly occur at lower speeds. This concept was not investigated.

Many of the speed-volume curves show a speed increase with volume to a peak speed at about 800 vph for the two-lane roadway. A possible explanation for this is that below this volume most drivers do not seem to be affected by other traffic. The driver chooses his own speed, which is below the peak speed. Above this volume, more of the traffic is restricted, and the average speed is lowered.

Mixed Volume-Equivalent Passenger Car Volume Graph

After correcting each curve for maximum volume, combining data from the time interval and platoon methods, and determining the family of speed-volume curves for all truck percent groups, the mixed volume-equivalent passenger car volume graph (Fig. 5) was drawn. To find the equivalent passenger car volume, the mixed volume is entered, its intersection with the desired truck percentage group curve is found, and the coordinate is read. For example, using a mixed volume of 3,000 vph, the equivalent passenger car volumes in vph are 3,000 (0 percent trucks), 3,300 (20 percent trucks), and 3,800 (40 percent trucks). This graph has certain limitations. No relationships are shown for mixed volumes below 1,000 vph. Also, since the equivalent passenger car volume is based on average speed, which reaches a maximum for each truck percentage group, no relationship can be shown for speeds greater than the maximum average speed attained by each truck percentage group.

Truck Equivalent Factor Curves

The approximate truck equivalent factors, as computed for the speed range of 35 to 43 mph, are as follows:

Truck Percentage Group	Factor
20	1.60
40	1.65
60	1.75
80	1.90

These factors are affected exponentially by the leveling off of speeds between 43 and 46 mph. However, this leveling off occurs in the lower volume region where truck factors on 50-mph roads may not be meaningful.

CONCLUSIONS

For the dual-dual roadway under the conditions studied, a relationship between mixed volume and equivalent passenger car volume was determined for 20, 40, 60 and 80 percent truck groups. At a constant speed, a mixed volume was assumed to be equivalent to the corresponding express roadway volume. As would be expected, this relationship shows that, for a constant mixed volume and with increasing truck percent, the equivalent passenger car volume increases and average speed decreases.

A truck appears to be equivalent to less than two cars.

The best fit second-degree speed-volume curves for several truck percentage groups showed an increase in speed with volume to a peak speed at 800 vph for a two-lane road and then the expected speed decrease. Below this volume there is apparently little restriction of movement and the driver chooses a speed below the peak speed. Above this volume traffic restriction increases and lower speeds result.

Various errors occurred in recording and coding the data. Sometimes a vehicle was not recorded at both ends of the speed course. Thus, speeds were not available for every vehicle. A few errors may have occurred in manually classifying vehicle type.

If, in coding the data, an undetected passing maneuver occurred, or if an incorrect pair of "blips" on the recorder chart were used, or if the time difference were measured incorrectly, then the speed was also incorrect. A small error in the time difference results in a larger error at higher speeds than at lower speeds. Errors in coding chronological time were detected by computer programs as headway errors.

Coding the data for this study was an enormous job. This method is not practical for large quantities of data or headway data for more than two lanes. With more than two lanes, overlapping pneumatic tubes are required, and interpreting the recorder chart becomes more complicated.

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Appendix

TABLE A1
VOLUME-SPEED COMPILATION (15-SECOND TIME INTERVAL METHOD)

		Express Cars Only			Local Roadway (Truck Percent Groups) 0 - 5%			5 - 30%			30 - 50%			50 - 70%			70 - 90%*			90 - 100%		
Size[1]	Vol[2]	#[3]	Spd[4]	SE[5]	#	Spd	SE	#	Spd	SE	#	Spd	SE	#	Spd	SE	#	Spd	SE	#	Spd	SE
3	720	200	46	4.1	205	47	4.9	-	-	-	233	46	4.6	168	44	4.7	114	44	4.8	83	43	5.1
4	960	186	47	4.1	155	49	4.5	206	46	4.5	-	-	-	180	45	4.5	65	44	4.1	44	44	4.5
5	1200	163	47	4.0	109	48	4.5	147	47	4.3	152	45	4.9	109	44	4.7	35	44	4.1	21	43	5.7
6	1440	135	46	3.4	75	48	4.3	105	47	4.3	99	46	3.6	164	45	3.8	54	43	3.9	19	45	4.2
7	1680	122	46	3.5	46	47	4.9	145	46	3.7	80	46	3.9	62	45	4.6	27	44	3.2	3	44	0.7
8	1920	106	46	3.3	20	48	3.8	87	45	4.5	49	45	3.7	81	44	3.0	9	43	2.8	2	43	0.4
9	2160	70	46	3.0	18	46	4.5	52	45	4.3	65	44	4.4	42	42	4.4	14	44	4.7	-	-	-
10	2400	79	46	3.6	6	46	2.2	33	46	4.5	36	45	4.8	27	43	3.5	4	45	5.1	-	-	-
11	2640	51	45	2.6	-	-	-	35	43	4.7	28	43	3.3	15	41	3.9	-	-	-	-	-	-
12	2880	54	45	3.0	-	-	-	14	43	3.9	14	40	3.7	13	43	2.7	-	-	-	-	-	-
13	3120	41	44	3.1	-	-	-	9	44	3.3	15	39	4.0	2	38	3.5	-	-	-	-	-	-
14	3360	41	43	3.6	-	-	-	-	-	-	6	37	6.8	5	42	6.0	-	-	-	-	-	-
15	3600	24	42	3.6	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
16	3840	24	42	3.3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
17	4080	22	42	3.6	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
18	4320	10	41	3.1	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
19	4560	2	39	1.1	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
20	4800	6	38	2.5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
21	5040	3	40	0.9	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
TOTAL		1339			634			833			777			868			322			172		

[1]Size: Veh./15-Sec

[2]Vol: Volume (vph)

[3]#: Number of observations

[4]Spd.: Average Speed (mph)

[5]SE: Standard Error (mph)

*Note: These values resulted in a concave curve. This line was not used.

TABLE A2
VOLUME-SPEED COMPILATION (30-SECOND TIME INTERVAL METHOD)

		Express Cars Only			Local Roadway (Truck Percent Groups)																	
					0 - 5%			5 - 30%			30 - 50%			50 - 70%			70 - 90%			90 - 100%		
Size[1]	Vol[2]	#[3]	Spd[4]	SE	#	Spd	SE	#	Spd	SE	#	Spd	SE	#	Spd	SE	#	Spd	SE	#	Spd	SE
3	360	79	46	4.5	54	46	4.2	-	-	-	60	46	4.5	38	44	4.7	-	-	-	14	42	5.1
4	480	71	46	4.0	41	48	4.4	45	47	4.3	-	-	-	46	44	4.3	29	42	3.6	7	45	2.4
5	600	73	46	3.5	38	49	4.1	76	47	4.4	59	45	4.7	38	43	4.6	25	44	3.6	5	45	5.6
6	720	74	47	3.7	32	48	3.8	52	48	4.7	46	46	2.9	74	44	4.0	16	42	3.6	2	45	0.0
7	840	63	46	3.5	29	48	2.7	84	46	4.2	46	45	3.8	28	44	4.2	35	43	3.0	3	48	2.1
8	960	63	46	3.5	11	52*	2.4	65	47	3.5	38	45	3.7	54	44	3.1	26	43	4.6	2	44	0.8
9	1080	53	47	3.6	7	48	3.2	68	46	3.9	68	45	3.9	38	45	3.2	11	45	4.3	2	43	5.4
10	1200	47	46	2.7	15	46	4.6	49	46	4.2	55	47	2.9	47	45	3.4	27	44	3.6	4	43	3.2
11	1320	46	46	3.0	4	47	3.2	61	47	3.9	43	47	3.3	28	44	2.8	10	46	3.2	2	44	0.5
12	1440	47	46	3.0	5	47	2.5	51	47	4.1	36	45	4.4	33	45	2.9	9	42	2.8	2	44	0.5
13	1560	40	45	2.7	5	48	3.1	33	47	2.7	47	44	2.9	35	43	3.6	8	43	2.7	-	-	-
14	1680	37	45	2.8	3	47	2.8	28	45	2.3	26	44	4.1	19	45	3.7	7	42	2.8	-	-	-
15	1800	29	44	4.2	-	-	-	30	46	3.8	25	45	3.2	19	42	3.4	4	42	2.2	-	-	-
16	1920	25	46	2.6	-	-	-	24	46	3.5	31	43	3.4	18	43	3.5	3	42	1.9	-	-	-
17	2040	18	46	2.0	-	-	-	13	45	3.3	18	42	3.9	6	41	2.9	4	43	0.9	-	-	-
18	2160	18	45	3.4	-	-	-	15	46	3.8	15	43	4.1	4	45	1.9	2	36	1.5	-	-	-
19	2280	24	46	3.2	-	-	-	5	45	4.7	10	44	2.5	5	45	4.7	-	-	-	-	-	-
20	2400	15	44	1.8	-	-	-	3	45	4.2	11	43	2.3	6	43	3.2	-	-	-	-	-	-
21	2520	14	45	1.4	-	-	-	6	44	4.9	3	41	2.7	3	42	1.8	-	-	-	-	-	-
22	2640	9	44	3.3	-	-	-	4	43	3.2	3	45	1.6	-	-	-	-	-	-	-	-	-
23	2760	8	45	3.0	-	-	-	-	-	-	6	41	4.4	-	-	-	-	-	-	-	-	-
24	2880	16	44	3.3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
25	3000	6	44	2.7	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
26	3120	6	44	3.3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
27	3240	9	44	1.7	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
28	3360	4	43	1.4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
29	3480	3	42	5.1	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
30	3600	11	42	2.6	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
31	3720	4	40	2.7	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
TOTAL		912			244			712			646			539			216			43		

[1]Size: Veh./30-Sec.
[2]Vol: Volume (vph)
[3]#: Number of observations
[4]Spd: Average Speed (mph)
[5]SE: Standard Error (mph)
*This value seems too high and was not used in the analysis.

TABLE A3
VOLUME-SPEED COMPILATION (PLATOON METHOD)

	Express Cars Only			Local Roadway (Truck Percent Groups) 0 - 5%			5 - 30%			30 - 50%			50 - 70%			70 - 90%			90 - 100%		
Vol[2]	#[3]	Spd[4]	SE[5]	#	Spd	SE	#	Spd	SE	#	Spd	SE	#	Spd	SE	#	Spd	SE	#	Spd	SE
650	-	-	-	1	48	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
750	-	-	-	-	-	-	1	46	-	-	-	-	1	47	-	-	-	-	-	-	-
850	-	-	-	1	46	-	-	-	-	1	46	-	-	-	-	-	-	-	-	-	-
950	-	-	-	-	-	-	-	-	-	1	47	-	3	42	6.6	-	-	-	-	-	-
1050	2	46	0.7	-	-	-	3	42	2.5	-	-	-	3	44	5.5	1	45	-	-	-	-
1150	1	45	-	1	49	-	4	47	3.5	2	48	0.7	3	44	3.0	1	44	-	-	-	-
1250	4	48	5.3	-	-	-	7	44	6.0	2	46	2.8	6	44	4.5	2	43	4.9	-	-	-
1350	5	46	3.5	3	43	3.0	2	46	2.8	5	44	4.2	2	46	4.2	6	44	2.2	-	-	-
1450	3	47	1.7	1	49	-	4	48	5.0	11	45	3.3	10	44	3.9	3	38	2.0	-	-	-
1550	4	46	1.2	3	49	2.6	12	47	4.3	8	47	3.3	10	46	3.4	2	45	0.7	-	-	-
1650	9	45	1.8	4	48	2.1	12	46	2.5	4	45	2.1	6	46	3.9	6	43	2.3	-	-	-
1750	4	51	2.5	2	49	0.7	13	48	3.1	13	46	2.8	9	43	1.6	7	43	2.7	-	-	-
1850	8	46	1.8	2	47	2.8	9	44	3.8	10	44	2.9	10	41	3.6	2	48	5.6	-	-	-
1950	11	47	3.8	-	-	-	9	46	2.5	12	44	3.2	14	42	4.1	3	43	4.9	-	-	-
2050	7	45	2.8	2	51	0.0	9	46	3.8	11	43	4.0	14	43	2.7	4	41	3.3	-	-	-
2150	17	46	2.5	-	-	-	13	47	2.9	9	45	3.0	11	44	4.5	1	46	-	-	-	-
2250	15	45	2.3	-	-	-	8	46	4.9	14	45	3.0	3	41	4.0	1	40	-	-	-	-
2350	15	44	1.8	1	48	-	11	47	3.4	14	42	2.6	8	43	3.9	2	40	7.0	-	-	-
2450	10	46	2.7	-	-	-	11	43	4.0	6	45	2.7	8	39	5.3	4	44	2.9	-	-	-
2550	14	44	3.1	1	47	-	10	46	3.6	8	43	3.6	7	43	4.0	1	40	-	-	-	-
2650	14	46	2.5	1	43	-	8	44	2.2	6	43	3.8	5	43	5.1	1	43	-	-	-	-
2750	4	46	2.2	-	-	-	5	46	3.3	7	41	3.3	2	44	4.9	-	-	-	-	-	-
2850	15	46	2.9	-	-	-	6	46	5.3	3	40	2.0	4	36	3.7	1	35	-	-	-	-
2950	7	43	3.5	-	-	-	2	43	1.4	2	44	1.4	3	40	5.5	-	-	-	-	-	-
3050	3	42	1.1	-	-	-	3	44	4.5	2	43	3.5	-	-	-	-	-	-	-	-	-
3150	5	46	3.1	-	-	-	2	41	1.4	3	42	4.1	-	-	-	-	-	-	-	-	-
3250	10	43	2.1	-	-	-	2	44	5.6	3	41	3.4	-	-	-	-	-	-	-	-	-
3350	9	42	3.3	-	-	-	2	44	1.4	2	42	2.1	-	-	-	-	-	-	-	-	-
3450	8	43	2.5	-	-	-	1	43	-	-	-	-	-	-	-	-	-	-	-	-	-
3550	11	41	3.2	-	-	-	-	-	-	1	35	-	-	-	-	-	-	-	-	-	-
3650	2	44	6.3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
3750	6	42	1.2	-	-	-	-	-	-	1	42*	-	-	-	-	-	-	-	-	-	-
3850	6	42	5.1	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
3950	1	45	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
4050	2	39	2.8	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
4150	1	40	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
4250	2	40	0.7	-	-	-	-	-	-	1	42*	-	-	-	-	-	-	-	-	-	-
TOTAL	235			23			169			161			142			48					

[2]Vol: Volume (vph) [3]#: Number of observations [4]Spd: Average Speed (mph) [5]SE: Standard Error (mph)

*Not used.

TABLE A4
SPEED-VOLUME CURVES (UNCORRECTED)

$Y = A X^2 + B X + C$
Y = Speed (mph)
X = Volume (vph)

Method		Truck Percent				
15 Sec.*	Express	0.0	20	40	60	80
A	-2.45×10^{-2}	-1.01×10^{-1}	-2.27×10^{-2}	-1.24×10^{-1}	-2.57×10^{-2}	-**
B	1.21×10^{-1}	1.04×10^{0}	-1.36×10^{-2}	1.36×10^{0}	7.24×10^{-3}	-
C	4.66×10^{1}	4.54×10^{1}	4.71×10^{1}	4.24×10^{1}	4.50×10^{1}	-
RMS	0.67	0.61	0.78	0.69	1.28	-
30 Sec.						
A	-5.93×10^{-7}	-1.40×10^{-6}	-5.56×10^{-7}	-5.63×10^{-7}	-6.81×10^{-7}	-2.58×10^{-6}
B	1.11×10^{-3}	2.46×10^{-3}	3.86×10^{-4}	7.36×10^{-5}	1.45×10^{-3}	6.07×10^{-3}
C	4.56×10^{1}	4.67×10^{1}	4.70×10^{1}	4.59×10^{1}	4.34×10^{1}	4.01×10^{1}
RMS	0.70	0.89	0.62	1.16	1.11	1.12
Platoon						
A	$-6.76 - 10^{-7}$	-2.33×10^{-7}	-1.30×10^{-6}	-8.63×10^{-7}	-9.09×10^{-7}	-3.62×10^{-6}
B	1.58×10^{-3}	7.34×10^{-3}	4.61×10^{-3}	1.04×10^{-3}	1.07×10^{-3}	1.14×10^{-2}
C	4.56×10^{1}	4.25×10^{1}	4.20×10^{1}	4.61×10^{1}	4.46×10^{1}	3.49×10^{1}
RMS	1.52	2.04	1.47	1.50	1.91	2.50

Note: RMS = Root mean square value.
*"X" in vehicles/15 seconds.
**This curve was concave and was not used.

TABLE A5

ASSUMED VOLUMES

Speed (mph)	Express*	Truck Percent 20	40	60	80*
35.0	4000	3580	3170	2750	2330
36.0	3860	3460	3060	2660	2260
37.0	3710	3320	2940	2560	2180
38.0	3550	3180	2820	2460	2090
39.0	3380	3030	2690	2340	2000
40.0	3190	2870	2540	2220	1890
41.0	3000	2690	2380	2080	1770
42.0	2780	2490	2200	1910	1620
43.0	2540	2260	1980	1700	1420
43.7	(2360)	(2040)	(1720)	(1390)	(1070)
44.0	2256	(1900)	(1600)	(1240)	-
44.3	(2180)	(1770)	(1440)	(1000)	-
45.0	1903	(1400)	(940)	-	-
45.7	(1600)	(860)	-	-	-
46.0	1358	-	-	-	-

Note: Volumes for 20%, 40%, and 60% trucks are assumed to fall equidistantly between the express and 80% truck sets of values. These volumes in parentheses are taken from the "Speed-Volume Curves, Fig. 4."

*Limiting values.

TABLE A6

TRUCK EQUIVALENT FACTORS (car/truck)

Speed (mph)	Truck Percent 20	40	60	80
35.0	1.58	1.66	1.76	1.89
36.0	1.58	1.65	1.75	1.88
37.0	1.57	1.65	1.74	1.88
38.0	1.57	1.64	1.74	1.87
39.0	1.57	1.64	1.74	1.86
40.0	1.57	1.64	1.73	1.86
41.0	1.57	1.64	1.74	1.86
42.0	1.58	1.66	1.76	1.89
43.0	1.62	1.70	1.82	1.98
43.7	1.78	1.93	2.16	2.51
44.0	1.94	2.02	2.36	-
44.3	2.16	2.28	2.97	-
45.0	2.80	3.56	-	-
45.7	5.30	-	-	-
Approx. Avg. (for speeds between 35 mph and 43 mph.)	1.60	1.65	1.75	1.90

Real-Time Evaluation of Freeway Quality of Traffic Service

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The freeway control process follows reasonably well-known laws that can be stated mathematically. The process can vary due to changes in flow characteristics, input constituents, economic factors, or policy decisions; but generally the control action to be exercised can be determined from the mathematics of the problem. Consequently, a freeway control system can permit a control computer to make decisions to select and implement control alternatives. This paper deals with the techniques and methodology needed to effectively control a freeway system in real-time.

The extent to which an optimum control policy can be established is directly related to the extent to which the pertinent traffic and control system variables can be measured and interpreted. Toward this idea, the report develops input-output analysis techniques that can be applied to a control environment. Using this analysis, a method of traffic block detection and location is presented and a step-by-step control action is developed.

With the application of real-time control to the problems of freeway congestion, the true science of freeway operation has emerged. The appealing idea of reacting to the problem as it happens can now be realized.

•INTRODUCTION of the electronic computer a little more than a decade ago fundamentally changed techniques of data processing. Work previously undreamed of soon became possible. Today, a second revolution is in progress, and its effect will ultimately be more rewarding than the first. Systems described as on-line and real-time are now being planned and installed. In these, data may be entered directly into the computer system from the environment in which they work and information sent back. To be precise, real-time refers to the performance of a computation during the actual time that the related physical process transpires so that results of the computations can be useful in guiding the physical process. This report deals with the application of this concept to the operation of an urban freeway.

As of now, several freeway surveillance and control projects are in the operational stage. The Chicago Area Expressway Surveillance Project on the Eisenhower Expressway, the John C. Lodge Freeway in Detroit, and the Gulf Freeway in Houston are the most notable, with other projects started in Seattle, Dallas, and Los Angeles. These control projects have worked toward the objective of real-time control of a freeway system, and to some degree the concept has been applied and its benefits realized. However, a single unified approach to freeway control has not yet been developed and applied on a real-time basis. It was to satisfy the need of developing the system requirements to perform the task of freeway control and traffic evaluation on a real-time basis that the Texas Transportation Institute was awarded a research contract by the U. S. Bureau of Public Roads.

TOWARD A SOLUTION

To achieve on-line, real-time control, a freeway control system must satisfy certain functional requirements. These requirements include (a) the ability to detect or

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sense the presence of vehicles at certain locations; (b) the ability to measure certain traffic and vehicle characteristics to the required accuracy; (c) the ability to predict ensuing conditions on a real-time basis; (d) the ability to determine a value for each control parameter; (e) the ability to determine and display, on a real-time basis, the state of the system and the ability to use this information as feedback to the control system; and (f) the ability to detect and react in real-time to transients in the traffic flow.

Several of these requirements invoke a data processing system in which at least one process is critically defined with regard to elapsed time (measured in fractions of a minute). When considering a freeway control system, the aggregate of the subprocesses must satisfy the critically defined deadline of releasing ramp vehicles for gaps—i. e., the critically timed process within the real-time system produces control outputs.

REAL-TIME SYSTEM FEATURES

To discuss a real-time evaluation scheme, several system components need to be defined (1). There are many more features than the ones to be considered here; however, only those applicable to a freeway control system are presented.

Input

Inputs from a freeway system arrive at the control computer at random intervals. These random inputs interrupt the computer and therefore must identify themselves. The number of input types affects the number and kind of formatting and editing programs, as well as computer storage allocations. An interrupt from a freeway lane detector requires servicing programs completely different from an interrupt indicating a green light confirmation from an on-ramp signal. Most inputs automatically deposit their data in temporary storage buffers to give the computer some leeway in servicing an interrupt. In the freeway control system described, the interrupts from the freeway and ramp detectors are handled in this manner to permit simultaneous detector interrupts to be serviced one at a time.

Software

Programs for real-time systems are constrained by the system deadlines. To keep the execution time short, some form of optimum programming is needed. Although useful for some processing routines, higher order languages, such as FORTRAN, are seldom acceptable for writing control programs because of compiler inefficiency. Real-time systems, therefore, incur the extra expense of machine-language coding for large portions of their programs. Also, time constraints result in program segmentation problems. When programs plus data requirements exceed main-storage capacity, the programs are broken up and stored in external storage units. This creates an access-time delay that must be added to the execution time of the program for estimating real-time throughput.

Reliability

Because of the urgency that justifies a real-time system in controlling a freeway, it seems evident that interruptions due to system failure cannot be tolerated. Thus, system specifications call for "fail-soft" operation (where some degradation of system performance is allowed, but total outages are not allowed). The goal of uninterrupted operation can be achieved by a simple, fixed-rate controller at the signal in the field. This would be a temporary control at a significantly degraded but safe level of performance.

REAL-TIME CONTROL SYSTEM

A real-time control system is one "which controls an environment by receiving data, processing them and returning the results sufficiently quickly to affect the functioning of the environment at that time" (2). The control system appears to the operator to be an integrated complex of equipment that monitors the environment, evaluates the environmental parameters, predicts environmental changes, and then makes optimal decisions for controlling the environment so that its changes stay within desired limits. The environment in this case is the freeway, service roads, and cross-street intersections. Even though the actual control is exerted on the on-ramps, the entire system of influence is considered.

Because the computer is simply one functional component of the system, control-system computers are usually specified to fit the initial system objectives exactly. This approach succeeds in freeway-control systems where the number of controlled on-ramps may expand and where the number of process control functions tends to remain constant.

The freeway control process follows reasonably well-known laws that can be stated mathematically. The process can vary due to changes in flow characteristics, input constituents, economic factors, or policy decisions; but generally the control action to be exercised can be determined from the mathematics of the problem. Consequently, a freeway control system can permit the control computer to make decisions to select and implement control alternatives. To do this and at the same time reduce the requirement for human operators in the system, the vehicle detectors and ramp control signals are automated and tied directly to the computer. Both the time constraints and the completeness of the freeway control algorithm relieve the control computer from storing a large data base needed to help make decisions. With the application of real-time control to the problems of freeway congestion, the true science of freeway operation has emerged. The requirements for the traditional extensive evaluation studies no longer exist. The appealing idea of reacting to the problem as it happens can now be realized.

ANALYSIS TECHNIQUES

Any analysis techniques used in a real-time environment must meet the following set of requirements:

1. They must be applicable to a fairly long section of freeway, perhaps several miles.
2. They must be capable of measuring the quality of traffic service in a system.
3. They must be adaptive to any freeway control site.
4. They must produce data suitable for feedback to the control models.

The prime example of an analysis technique that meets these requirements is the input-output analysis used by all three freeway surveillance and control projects. During the early Gulf Freeway studies, this technique was used in manual form by the Texas Transportation Institute (3). It is important that this analysis permit a close look at congestion formation and duration on any system of interest. To apply this scheme consider Figure 1, which shows a system in which the procedure may be used.

With the count stations as shown, the system would be the freeway within the count stations, or that area cordoned off by the count stations. Within this area, all of the following variables can be obtained for any time period:

1. Freeway input flow rate (V_i);
2. Freeway output flow rate (V_o);
3. Freeway flow rate by lanes;
4. Total and individual ramp input flow rate;

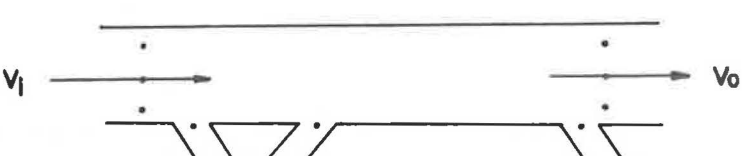

Figure 1. System used for input-output analysis.

5. Total and individual ramp output flow rate;
6. Total system travel time;
7. Total system travel;
8. Average speed in the system; and
9. Kinetic energy in the system (4).

QUALITY OF TRAFFIC SERVICE EVALUATION

With the use of a digital computer in the control of a freeway system, the traffic engineer can handle large volumes of data and operate in a real-time environment. Furthermore, in using a digital computer the operator has the flexibility to change the control philosophy, an asset he does not have when using a hard-wired or analog system.

The process control computer presently being used on the Gulf Freeway is an IBM 1800 Data Acquisition and Control System. The equipment available for input-output includes the following:

1443 Printer (150 lines per minute)
1442 Card Read-Punch (300 cards per minute)
2310 Disk Storage (storage device with both random and sequential access)
1627 Plotter
1816 Printer-Keyboard

Data Acquisition Software

To achieve and maintain responsive control of a process, it is necessary to continually sense the state of that process and to evaluate certain parameters and variables that describe the current operation of the process. This is the duty of the data acquisition program. As such, it is the subsystem of the control program or the feed-

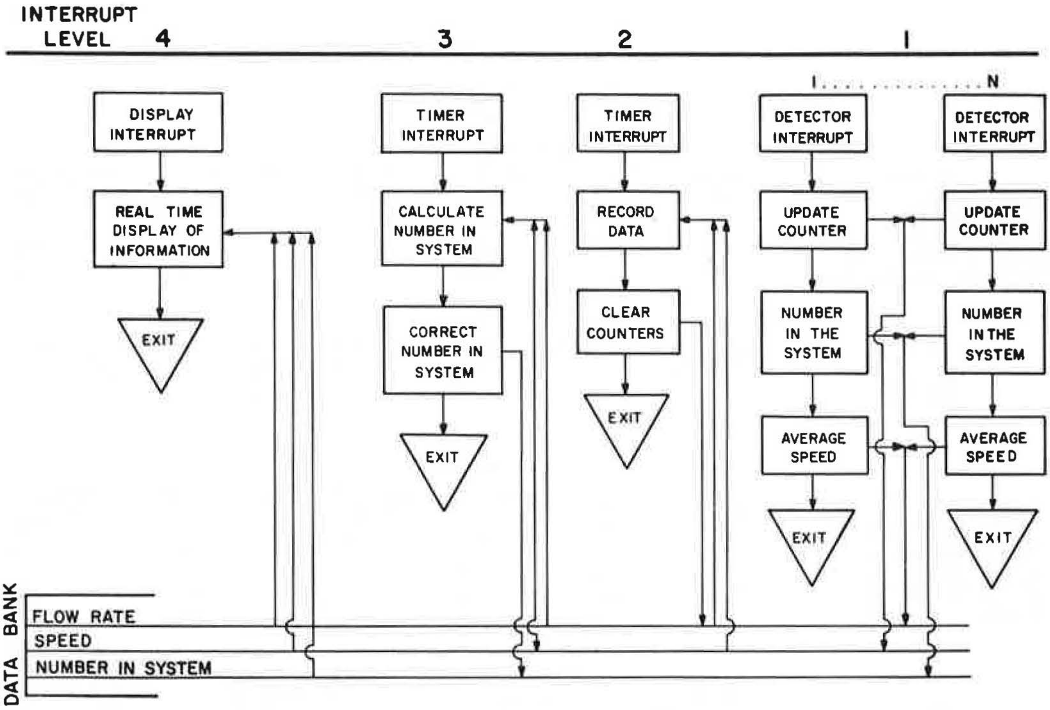

Figure 2. Logic diagram for data acquisition program.

back required for the optimal control of any process. The data acquisition program also serves to evaluate and display the state of the operation for the benefit of the operating agency and can provide a permanent record of operation for use in comparing different control strategies and for use in the development of new ones. The data acquisition subsystem further serves to detect equipment failures and provide the operator with an analysis of what effect the failure will have on the operation of the process.

Since the program is based on the concept of process control, several operations proceed in parallel. Figure 2 is the logic diagram for the program, with the interrupt levels listed at the top of the figure. The interrupt levels assign a priority to each section of the program. Using level one as the highest priority, an interrupt from the field, indicating the presence of a vehicle, will never be missed or delayed. Levels two and three operate from timer interrupts with the interrupt frequency specified by the operator. The frequency of level two is the frequency at which the flow rate is recorded. Level three is provided to check and correct errors introduced by the detectors in the field. This particular algorithm, using speed-volume-density relationships, calculates the number in the system so that any error introduced by the detectors will not be cumulative. The display interrupt, level four, can be manually or automatically set. Therefore, any desired interrupt frequency can be specified. The data bank at the bottom of Figure 2 is common to all levels of the program.

As an example of the information available for each subsystem, Table 1 gives the output for a 2-hour evaluation (6:30 to 8:30 a.m.) of two adjacent subsystems.

At any instant, the two basic traffic variables of volume and density are available for any subsystem. Figure 3 illustrates this aspect of the data acquisition program. The subsystem presented in this figure is 1 mile long, making the number in the system correspond to the system's vehicular density. Although the minute volumes are erratic, the variation is only random and does not indicate a breakdown in flow.

The information available through the data acquisition programs can also be used to determine the speed-volume-density relationships for each subsystem. The derived relationships for the Griggs-Telephone subsystem are shown in Figure 4. An

TABLE 1

SYSTEM CHARACTERISTICS AND PROCESS FEEDBACK FROM THE DATA ACQUISITION PROGRAM

SUBSYSTEM 1
DATE 2/ 2/68

LOCATION	630-830 VOLUME	700-800 VOLUME	PEAK HOUR	PEAK HOUR VOLUME	PEAK HALF-HOUR	PEAK HALF-HOUR VOLUME
FREEWAY AT 225 (3)	6577	3349	637	3766	652	1992
35 OFF RAMP (1)	177	73	732	111	801	65
225 ON RAMP (1)	1566	657	606	888	625	527
35 ON RAMP (1)	1427	640	618	780	629	456
WOODRIDGE OFF RAMP (1)	255	129	732	181	732	95
FREEWAY AT WOODRIDGE (3)	9137	4615	637	5033	640	2606

TIME	TOTAL TRAVEL (VEH-MI)	TOTAL TRAVEL TIME (VEH-HOURS)	KINETIC ENERGY (VEH-MI**2/HOUR)	AVERAGE SPEED (MPH)
630-830	4027.9	183.2	88529.6	21.9
700-800	2023.4	120.9	33866.9	16.7

SUBSYSTEM 2
DATE 2/ 2/68

LOCATION	630-830 VOLUME	700-800 VOLUME	PEAK HOUR	PEAK HOUR VOLUME	PEAK HALF-HOUR	PEAK HALF-HOUR VOLUME
FREEWAY AT WOODRIDGE (3)	9137	4615	637	5033	640	2606
WOODRIDGE ON RAMP (1)	611	307	721	359	752	195
MOSSROSE OFF RAMP (1)	146	53	733	94	757	59
MOSSROSE ON RAMP (1)	437	315	653	331	721	181
FREEWAY AT GRIGGS (4)	10037	5246	644	5473	644	2758
WAYSIDE OFF RAMP (1)	1060	466	733	560	620	308
FREEWAY AT BAYOU (3)	8977	4780	644	5028	652	2538

TIME	TOTAL TRAVEL (VEH-MI)	TOTAL TRAVEL TIME (VEH-HOURS)	KINETIC ENERGY (VEH-MI**2/HOUR)	AVERAGE SPEED (MPH)
630-830	9594.8	295.1	311896.0	32.5
700-800	4889.4	169.4	141123.4	28.8

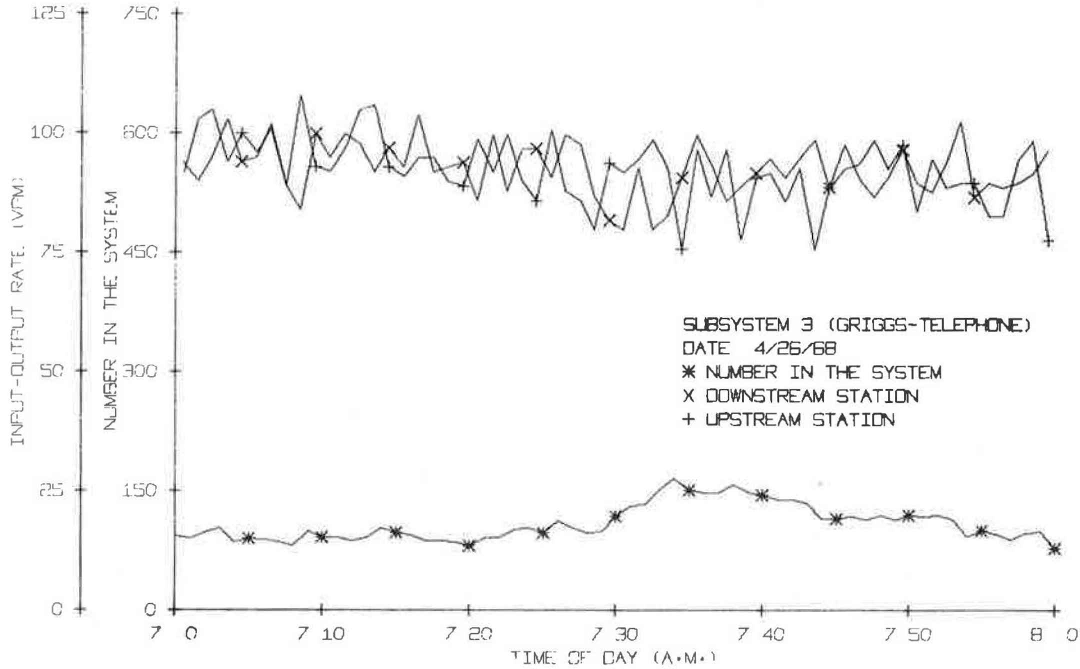

Figure 3. Real-time information provided by the data acquisition program.

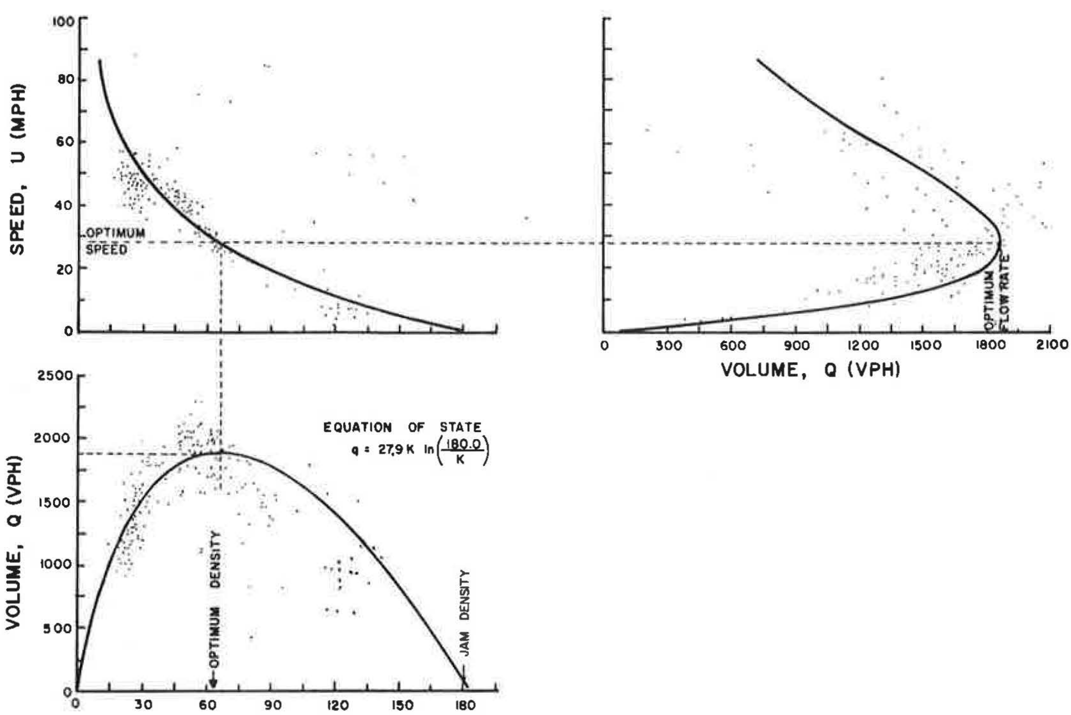

Figure 4. Volume-density-speed curves for Gulf Freeway inbound morning peak operation.

exponential model was used with the following equation:

$$q = ku_m \ln \frac{k_j}{k}$$

The parameters k_j and u_m were found through the use of a polynomial regression technique. The results of the regression were

jam density, k_j = 180.0 vehicles per mile
optimum speed, u_m = 27.9 miles per hour

and, from the curves in Figure 4, the following additional parameters can be found:

optimum concentration, k_m = 66.5 vehicles per mile
optimum volume, q_m = 1875 vehicles per hour

MULTILEVEL APPROACH TO SYNERGISTIC FREEWAY CONTROL

Drew et al (5) introduced a multilevel approach to the design of a freeway control system. They state, "The central idea behind this design is to share the effort of solution among two or more levels, each of which communicates both with the level directly above and that directly below."

The four levels of control, with the interaction between each level, are illustrated in Figure 5. Levels one and zero are controlled by the regulating function. This controller accomplishes what might be called the basic subgoal of the control system, which is optimal use of available gaps on the freeway and which is fulfilled by the timely release of ramp vehiches by the ramp signal.

The optimizing function adjusts the gap setting on the first level, regulating the controller in response to the outside-lane freeway operation (volume and speed) so as to maximize the ramp service volume. For example, if the setting on the regulating controller is too high, many gaps are left unfilled; if the setting is too low, many metered

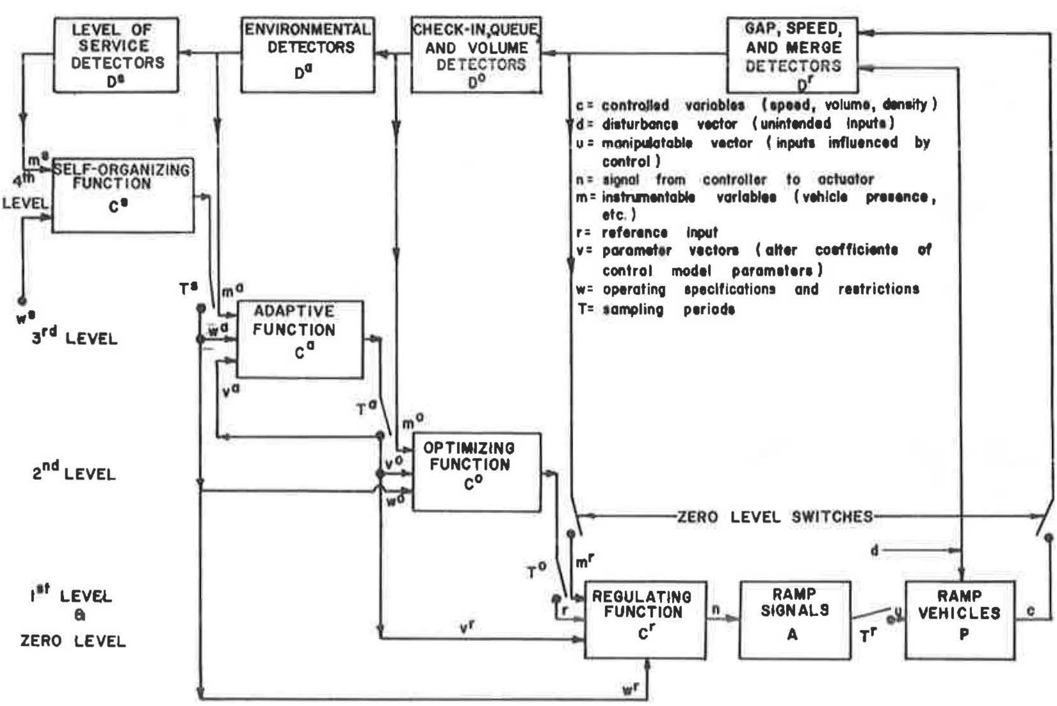

Figure 5. Decomposition of freeway control function.

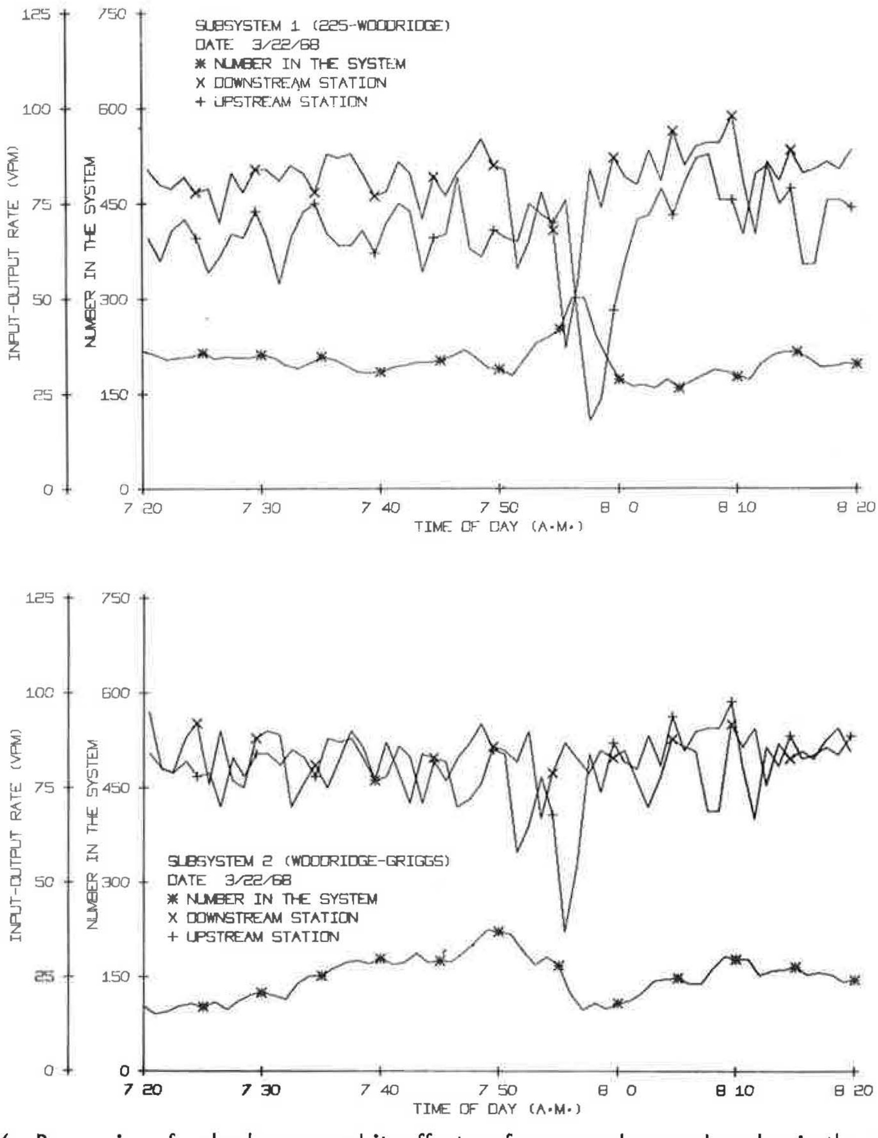

Figure 6. Progression of a shock wave and its effect on freeway volume and number in the system.

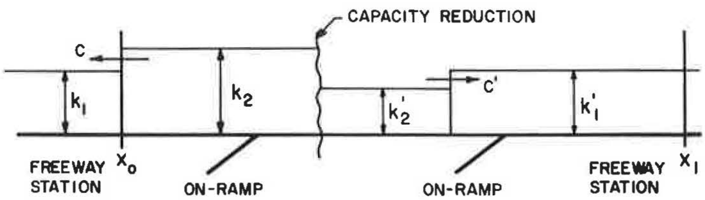

Figure 7. Stopping wave response to a reduction in capacity: k_2 density behind wave of stopping, k_1 and k_1' = density before block occurred, k_2' = density behind wave of clearing, c = velocity of stopping wave, and c' = velocity of clearing wave.

vehicles will reject the gaps and be forced to stop in the merging area where their presence will preempt metering. Both the regulating and optimizing functions are discussed by Brewer et al (6). The adaptive function and the self-organizing function deal with the freeway system and with the interaction between on-ramps.

The Adaptive Function

The function of the adaptive controller is to handle the unexpected system inputs, such as environmental factors and temporary capacity reductions. This third-level controller changes these transients in the traffic stream to inputs for the second- and first-level controllers.

Because of the devastating effects that a reduction in capacity has on freeway operation, a high priority must be given to detecting and locating this reduction when considering the system control of a freeway. This reduction could result from either an incident blocking one or more lanes or a bottleneck caused by a deficiency in design or a stalled vehicle on the shoulder. With the detection system in operation on the Gulf Freeway, capacity reductions and the resulting effects, such as shock waves, can be readily sensed.

Figure 6 shows the effect of a capacity reduction. Subsystem 2 clearly shows a shock wave moving through the subsystem. The shock wave reached the downstream freeway station at 7:57 a.m., with the block occurring about 1.5 miles downstream at 7:50 a.m. In Subsystem 2, it is of interest to note that, at the time when the wave crossed the upstream count station, its velocity was about 20 mph. The capacity reduction was caused by a stalled vehicle that momentarily blocked one lane of the freeway. Because there was such a high number of vehicles in the system before the stall, the downstream count station in Subsystem 3 did not show a reduction in flow rate.

Traffic Block Detection and Location

To reduce the effect of a capacity reduction on the operation of the freeway, it is important to devise a means of early detection. Some type of response to a reduction in capacity, such as regulating the upstream entrance ramp flow rates, must be made. Figure 7 is a graphic illustration of a stopping wave at freeway station X_0. At the

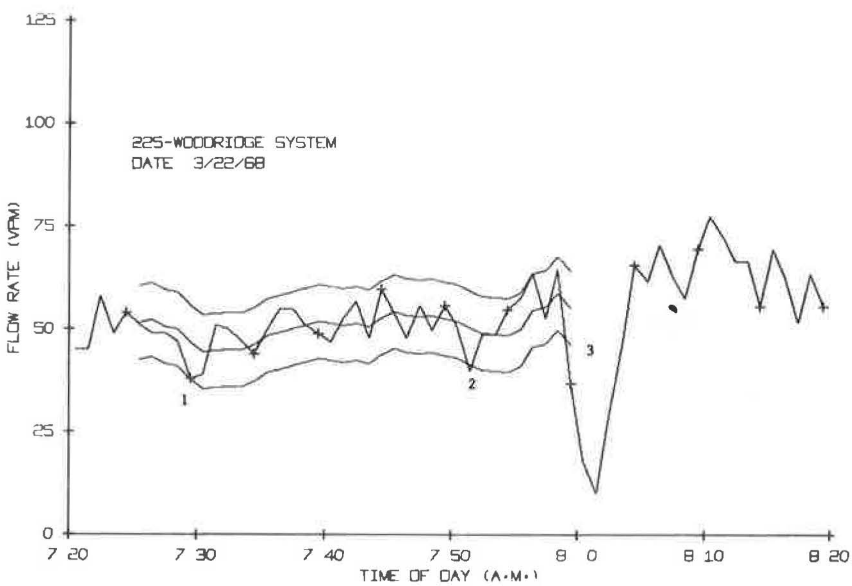

Figure 8. Recognition of a shock wave by digital computer.

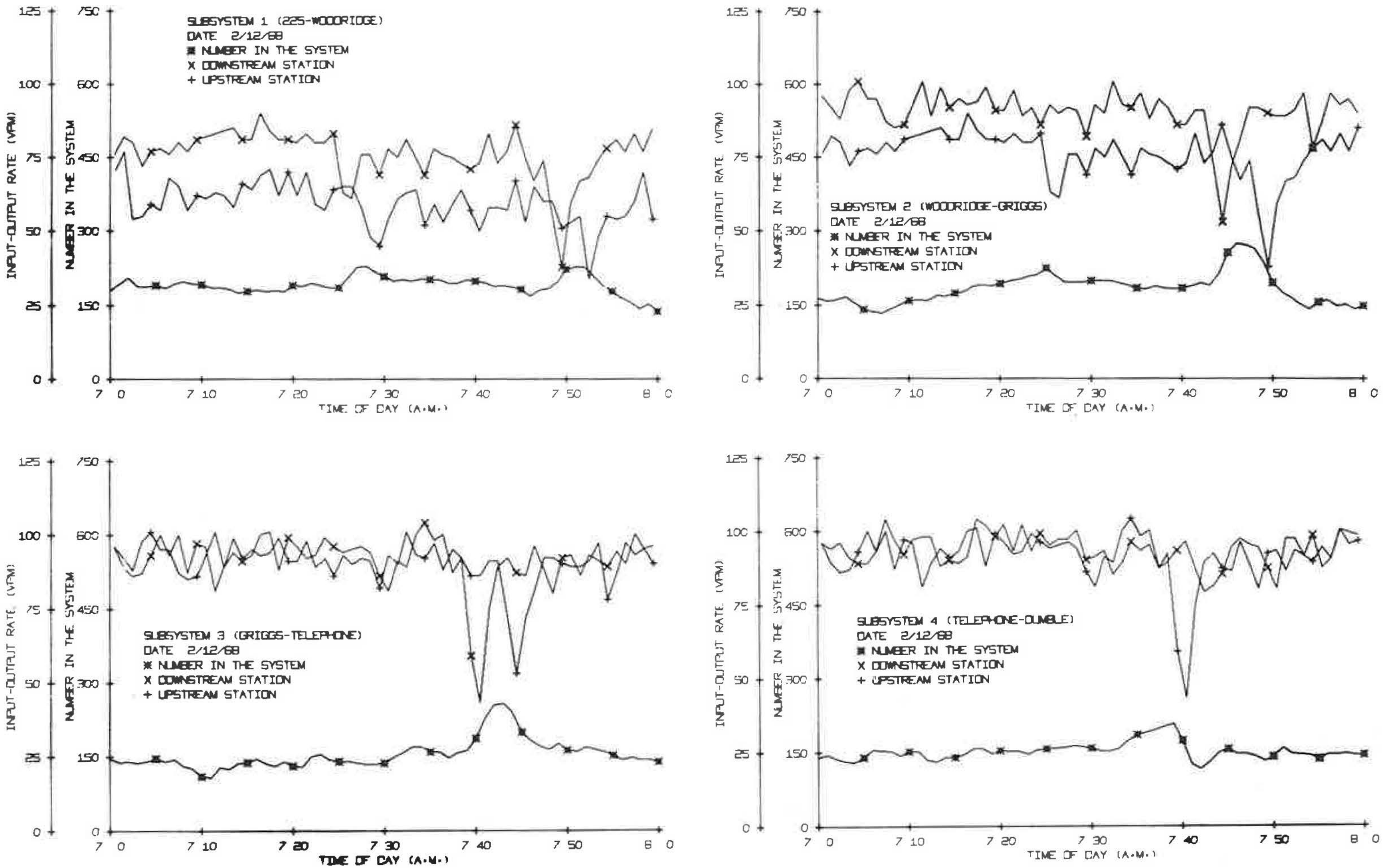

Figure 9. Effects of a capacity reduction on the operation of the Gulf Freeway control system.

position of the capacity reduction, the traffic on the freeway slowed down, and the stream immediately assumed a nearly saturated concentration of K_3. To the right of the capacity reduction, the density is reduced from K_1 (before the block occurred) to K_2', which in turn will reflect a higher mean traffic velocity. This differential in speed will be detected by the data acquisition program, locating the point of reduced capacity to within a few hundred feet.

Another method of shock wave recognition is illustrated in Figure 8. As mentioned before, some random variation of the flow rates over a count station is to be expected. To take this random variation into account, a running 5-minute average of the flow rate and corresponding upper and lower limits are plotted along with the 1-minute flow rates. The limits are two standard deviations away from the 5-minute average. As Figure 8 indicates, points 1 and 2 fell on or below the lower limit, but they remained there for less than 30 seconds. Point 3, however, fell well below the low limit for two consecutive 15-second periods. A decrease in the flow rate of this magnitude would indicate a capacity reduction of significant size. A comparison of the speed at this particular freeway station to the volume-density-speed curves would indicate either a stopping or clearing wave.

The block shown in Figure 7 could be caused by two different situations. The most common and frequent stoppage is a stalled vehicle or accident, either of which could affect the freeway a few seconds or several minutes. Shock waves can also be caused when the demand exceeds the capacity of a section. Figure 9 shows just such a situation. In Subsystem 4, the Telephone-Dumble section, there is a critical area in the vicinity of the Telephone on-ramp. The merging capacity in this area is about 500 vehicles per 5 minutes. The 5-minute flow rate just upstream of the merge area before the breakdown shown in Subsystem 4 was 515 vehicles. This is brought out only to show that, with some knowledge of the capacity of a freeway section under control, many breakdowns can be avoided. However, the process evaluation discussed in this paper is generalized to the point that the bottlenecks in the system do not have to be defined before control is started.

Control Action

With a capacity reduction detected and located, action should be taken to reduce the effect of the shock wave on the operation of the freeway and perhaps to dissipate the shock wave altogether. Brewer et al (6) presented a model that describes the service flow rate of a controlled entrance ramp in terms of the freeway flow rate and the service gap setting on the controller. The family of curves generated by the model is shown in Figure 10. This ramp-service flow rate model gives the ramp flow rates

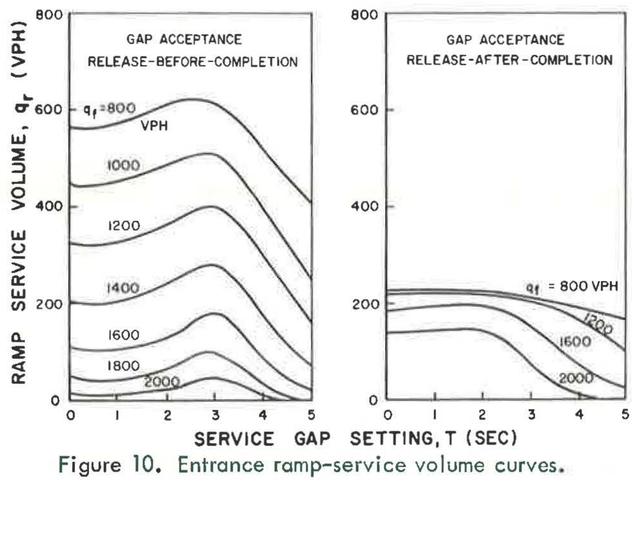

Figure 10. Entrance ramp-service volume curves.

that can be expected when a particular service gap is set on the controller. Therefore, the effect of a capacity reduction on freeway operation can be compensated for by increasing the service gap, thus decreasing the flow rates at the upstream ramps by the amount of the capacity reduction. A typical control example, using Figure 8 as the state of the system, might proceed as follows:

Step	Time	Description
1	7:59:45	The flow rate is below the lower limit, indicating a possible capacity reduction.
2	8:00:00	The flow rate has been below the limit for two consecutive 15-second periods, showing a significant capacity reduction.
3		At this point calculate the drop in flow rate over the last 1-minute period. For this example the flow rate for three lanes has been reduced by 30 vehicles per minute. The measured freeway flow rate, g_f, is 1,100 vph in the outside lane, and the current service gap, T_s, setting on the upstream ramps is 2.8 seconds.
4		Using Figure 10 and an intermediate type operation, the service gap setting on 8 of the upstream on-ramps is increased to 4.8 seconds to compensate for the first minute of the capacity reduction.
5		The outside-lane speed profile is now checked to locate the origin of the capacity reduction.
6	8:00:15	The freeway flow rate is again checked for the progress of the shock wave, and the control action is returned to step 3.

As the shock wave proceeds upstream and crosses each set of freeway detectors, the same procedure is repeated until the shock wave is dissipated or leaves the control section. When this happens, the control procedure would automatically return to the original service gap setting.

Self-Organizing Function

The fundamental property of a self-organizing, or learning, control system is its ability to perform better as time progresses. The fourth-level function can be programmed to automatically update the parameters used in the lower three levels of control. Capacity reductions offer an example of its application to third-level control. The capacities of a geometric bottleneck, an icy pavement, a wet pavement, etc., can

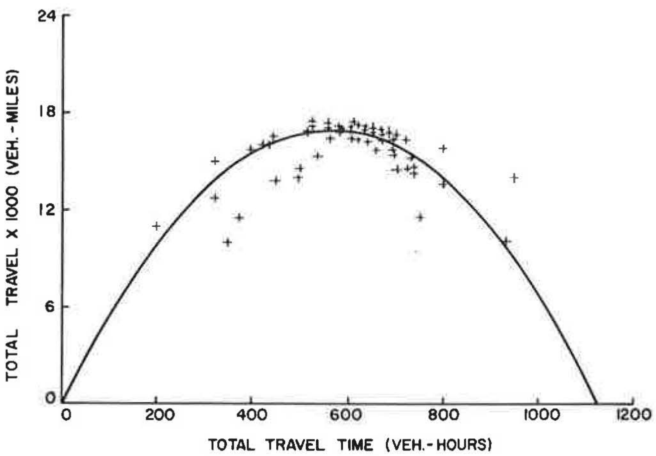

Figure 11. Operating characteristics for a 2½-mile section of the Gulf Freeway control system.

be "learned". Once the fourth-level computer has learned the capacity profile of the freeway, it will no longer allow the second level to exceed this capacity.

To judge whether 1 day's operation is better or worse than the previous average operation, a curve such as Figure 11 is also programmed into the self-organizing function. Using data collected from the control section, the self-organizing function could actually develop this curve, as was done for the Gulf Freeway inbound operation (7).

The self-organizing function determines what the decision vectors should be on the basis of measurable freeway characteristics and the intervention of the operation in the system. These decisions are based on the accumulated experience and understanding of the process being controlled, i.e., the freeway system. Furthermore, while they alter the control laws in the lower levels, these decisions are subject to the specifications, goals, and constraints embodied in the worth vector.

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