TRANSPORTATION RESEARCH RECORD 699

Highway Capacity and Level of Service

TRANSPORTATION RESEARCH BOARD

COMMISSION ON SOCIOTECHNICAL SYSTEMS
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY OF SCIENCES
WASHINGTON, D.C. 1979

Transportation Research Record 699
Price $3.40
Edited for TRB by Mary McLaughlin

mode
1 highway transportation

subject area
55 traffic flow, capacity, and measurements

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Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.
Highway capacity and level of service.

(Transportation research record; 699)
1. Traffic surveys–Congresses. 2. Traffic flow–Congresses.
3. Express highways–Congresses. I. Title. II. Series.
TE7.H5 no. 699 [HE369] 380.5'08s [388.4'131'4]
ISBN 0-309-02844-2 ISSN 0361-1981 79-607090

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Contents

Cost-Effective Level of Service and Design Criteria

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Research was initiated to determine the appropriate levels of service that could be applied to the design of rural highways in Alberta under a variety of conditions. By using Highway Capacity Manual speed-volume curves for two-lane rural roads and passenger-vehicle road user costs and average construction and maintenance costs for Alberta, cost-volume relations (unit average cost supply functions) are derived and applied in the analysis of annual hourly traffic data obtained at permanent counter locations in Alberta. To identify the demand function, determination of design hourly volumes from curves for the highest hour of the year is discussed (all 8760 hours are ranked). The supply and demand functions are then interrelated. Preliminary findings indicate that (a) a minimum unit cost per passenger vehicle per kilometer can be correlated with volume-capacity ratios depending on capital, maintenance, and user costs and (b) a design hourly volume K-factor based on the knee of the curve for the highest hour of the year is more consistent than the traditional 30th highest hourly volume K-factor. Although a methodology has been developed to determine a cost-effective volume-to-capacity ratio and the findings indicate that it may be appropriate to set a range of levels of service for different road characteristics, further work is required to refine cost relations so as to reflect terrain, traffic composition, and current road user costs.

In considering the planning and design of a new highway or improvements to an existing one, both supply and demand must be considered. On the supply side, three major factors are normally considered in designing a new facility or in evaluating an existing one: (a) safety, (b) structural adequacy, and (c) level of service (traffic congestion). On the demand side, the major factor considered is existing and future demand.

In practice, agencies try to design a facility to provide an acceptable level of service to the user. In the past, design criteria were adopted that provided a very high level of service and thus low levels of congestion. However, in recent years, because of the increasing costs associated with continuing to provide a relatively high level of service, agencies have had to reconsider their policies and associated design criteria.

Although supply and demand are often evaluated separately, they are interrelated. This paper offers a methodology for evaluating these relations based on concepts previously developed by Haritos (1), Cameron (2), and Winfrey and Zellner (3). We hope it makes a modest contribution toward clarifying and understanding some of the unresolved issues in transportation economics.

This paper first presents a methodology for deriving the most economical level of service for the supply side. Then, on the demand side, a design hourly volume and its selection are discussed. The paper concludes with the presentation of suggested design criteria.

MEASUREMENT OF ECONOMIC EFFICIENCY

Generally, highways are provided at public expense and little or no direct income is derived. Therefore, the agencies involved should design roadways not to maximize proft but to minimize costs to the public while providing good standards of safety and mobility.

In any attempt to define the costs attributable to providing a highway link, one might include costs for right-of-way, construction, maintenance, environmental disruption, motor-vehicle running costs, accidents, and travel time. In the analysis presented here for the Canadian province of Alberta, the following cost factors were used: (a) construction, (b) maintenance, (c) motor-vehicle running cost, and (d) travel time.

Although right-of-way costs can be a major factor, it has been assumed that right-of-way has full terminal value and therefore it is not considered here. Quantifiable costs related to environmental disruption—e.g., costs of erosion control, noise attenuation, and other measures to protect the environment—can be included in construction costs. However, unquantifiable costs, such as those for wildlife disruption, are not included. Accident costs have not been included here because no Alberta data were readily available and because accident costs can be considered part of the safety analysis that some agencies prefer to handle separately.

Fixed capital costs for roads are high, and annual maintenance costs are often also significant. If the road carries little traffic, the unit cost of providing the roadway is very high; as volume increases, however, unit cost decreases.

For road user costs (time plus running costs), lower traffic volumes usually provide the least unit cost and, as volumes increase, the cost to the user increases because of congestion. These relations are shown in Figure 1. Merging these two curves should result in a relationship in which, at some volume of traffic, a minimum cost of travel will occur.

To compute this relationship, it is necessary to relate capital and maintenance costs and road user costs to a common base. Since capital and maintenance costs are a function of volume and road user costs are a function of speed, the speed-volume relations presented in the 1965 Highway Capacity Manual (HCM) (4) were used to determine costs in dollars per vehicle kilometer. The following values were used:

1. Capital cost: \$181 250/km,
2. Maintenance cost: \$1000/km,
3. Discount rate: 8 percent over 15 years, and
4. Road user costs: 1976 Alberta running costs (5) for 100 percent passenger vehicles on level tangent sections and a value of time of \$3.70/h/passenger vehicle.

Capital cost was brought back to an equivalent uniform annual cost, and annual maintenance cost was added. This cost was then brought down to an average hourly cost and divided by the volume of vehicles for given speeds obtained from the speed-volume curves.

Vehicle running costs for Alberta are empirically derived values presented in tabular form that give the cost to run a vehicle at various speeds. Travel time costs were simply divided by the desired speeds to obtain the cost to travel 1 km at that speed and were added to vehicle running costs. Combining these costs resulted in the curve shown in Figure 2, which indicates that total unit cost minimizes at a volume-to-capacity (V/C) ratio of approximately 0.28 (level of service B) for passenger vehicles on level tangent sections.

Although this lends some credibility to providing level of service B as a design criterion, there are several factors that will affect the analysis and cause a shift of the minimum cost point: much higher costs as a result of constructing a roadway in difficult terrain; increased running costs on grades and curvatures; the effect of trucks, buses, and recreational vehicles in the traffic stream; and the value of time.

Figure 1. Road user and capital costs versus traffic volume.

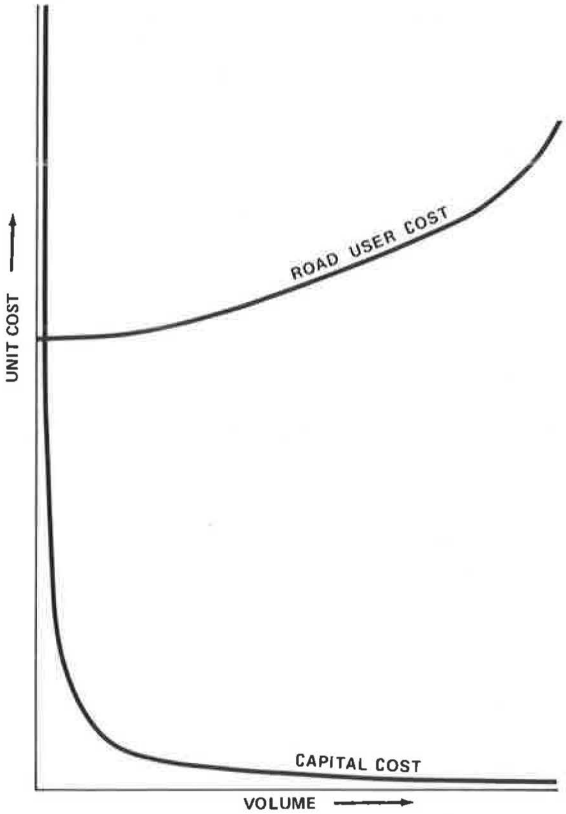
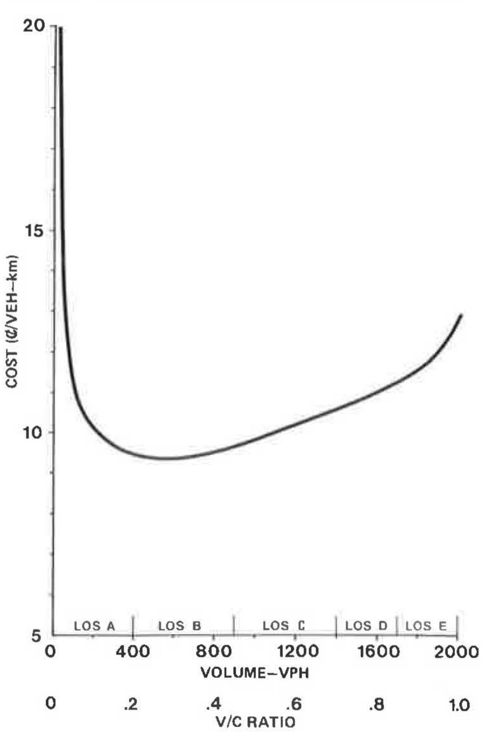


Figure 2. Combined cost versus levels of service.



SENSITIVITY TESTING

To reflect varying topography, traffic composition, and

Figure 3. V/C ratio cost minimas versus value of time.

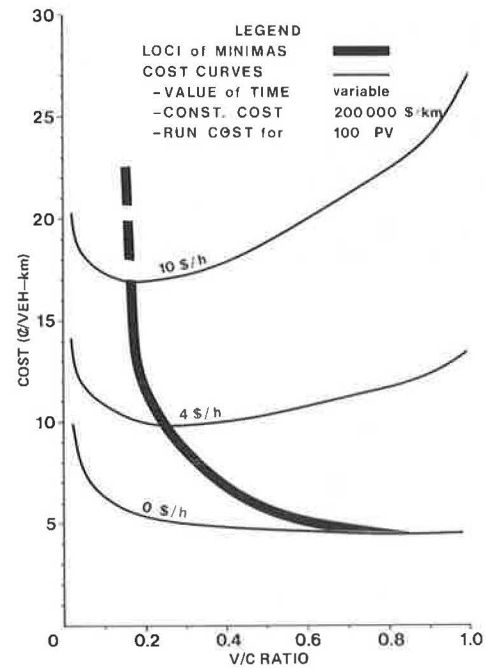
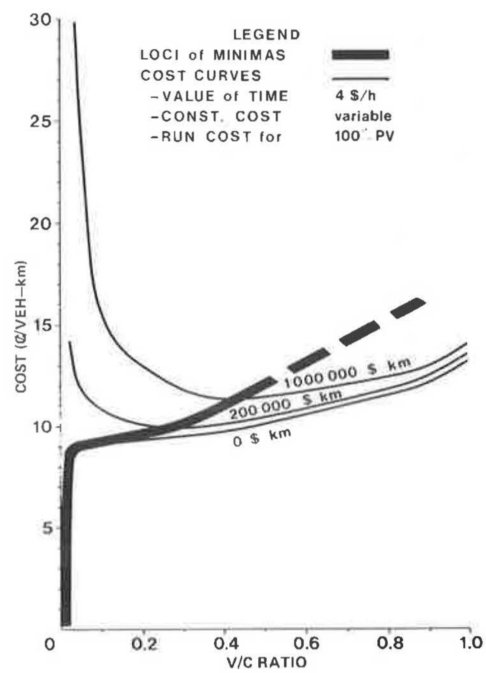


Figure 4. V/C ratio cost minimas versus construction cost.



time values, each cost parameter was varied while the others were held constant. The value of time was varied from \$0 to \$10/h, construction costs from \$0 to \$1 million/km, and running costs from \$0.043 to \$0.063/vehicle-km (as represented by a vehicle composition that varies from 100 to 60 percent passenger vehicles with 20 percent trucks and 20 percent recreational vehicles).

Figures 3-5 show the plots of the loci of the minimas for each varying parameter. Whereas both increased running costs and construction costs tend to shift the point of minimization toward the lower level of service, the value of time is the most sensitive variable—especially for the lower time values—and tends to shift the point of minimization toward the higher level of service with increasing values. This testing indicates that no single V/C ratio can be defined as the most economical and suggests instead a level-of-service range that depends on topography, traffic composition, and trip purpose.

Although the technique presented here provides an economical V/C ratio, it is inherent in the calculations that uniform hourly volumes occur for every hour of the year. Since this is not the case in reality, hourly, daily, weekly, and seasonal variations were investigated by considering (a) daily traffic volumes for two permanent counter locations in the Alberta Primary Highway System and (b) average annual daily traffic (AADT) for 29 permanent counter locations in the province.

The procedure followed was to calculate the total cost for each hour of the year based on the cost-volume relation shown in Figure 2, accumulate these costs for each day and for the year, and then divide by the daily or annual volumes. Each cost-volume data pair was then plotted, and a hand-fit curve was drawn.

Figures 6 and 7 show the daily cost-volume relations generated and show that there is a leveling off of unit costs when the daily volumes approach 7000-8000 vehicles/d. Although minimization is not clearly evident, it appears that minimization occurs at approximately 8000 vehicles/d in Figure 7.

Figure 8 shows the AADT cost-volume relationship. Note that minimization does not occur within the range of available two-lane data. To extend the curve, selected multilane counters were analyzed. Because it was assumed that the volumes carried on these multilane facilities could be accommodated on a two-lane roadway, the costs were calculated by using two-lane capacities. Although there are insufficient data to plot a curve so as to determine the point of minimization with confidence, the same leveling-off trend as that found in Figures 6 and 7 is observed. If minimization did occur, it would not be expected before 8000 vehicles/d (Figure 7). It appears that the traveling public in rural Alberta would not accept this level of service as satisfactory. In fact, Provincial

Figure 5. V/C ratio cost minimas versus vehicle running cost.

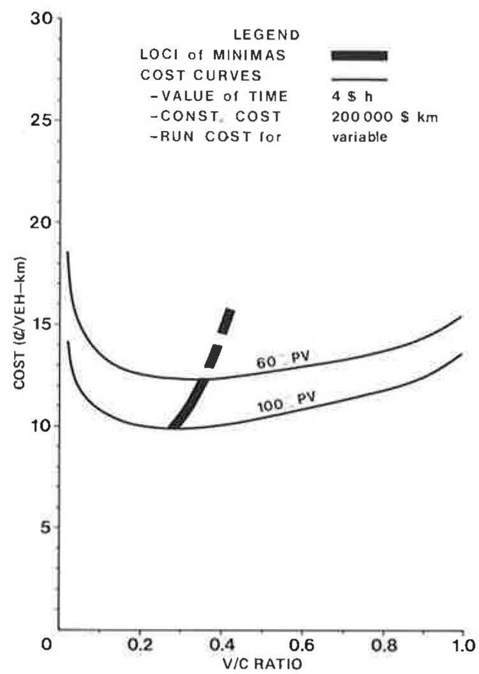
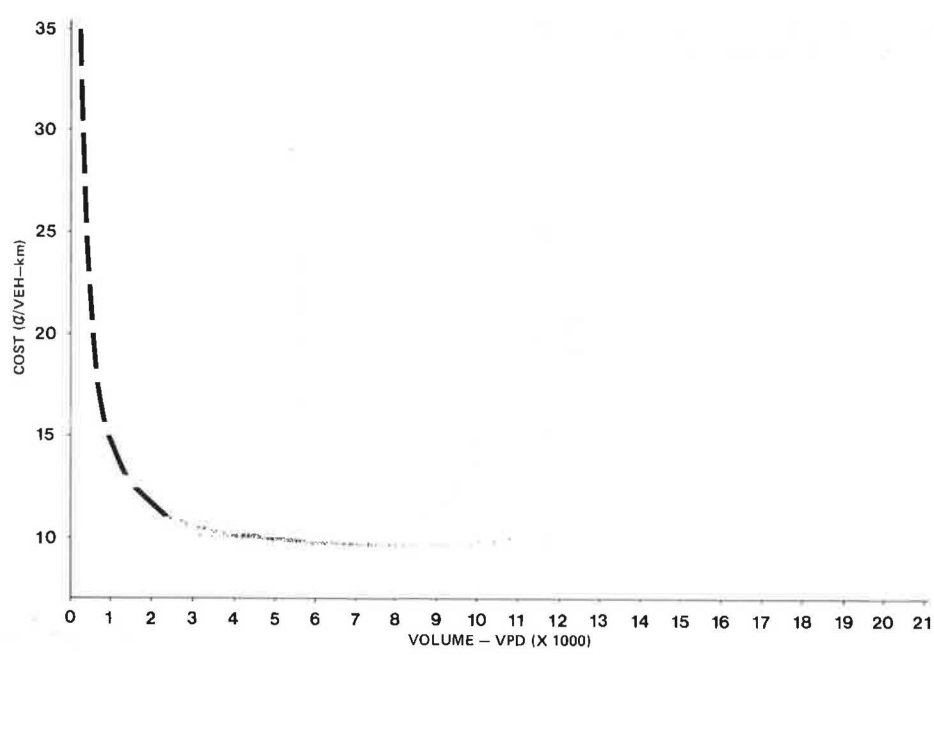


Figure 6. Daily cost-volume relations at counter 63.



Highway 11 from Red Deer to Sylvan Lake (counter 63 shown in Figure 6), which has an AADT of 5600 vehicles/d, is being proposed for upgrading (possibly four lanes or an alternate two-lane route) within the next three years.

The preceding technique can be used to determine the most cost-effective V/C ratio for any given set of circumstances in analyzing the need to upgrade a highway facility or provide an entirely new route. There remains, however, the unresolved issue of relating the cost-effective V/C ratio, determined on an hourly basis, to the usual practice of using a design hourly volume, which is often expressed as a percentage of AADT. This paper attempts to define a rational approach to determining and correlating the relations.

DESIGN HOURLY VOLUME

The design hourly volume (DHV) is the volume of traffic during 1 h that is used as an acceptable operating condition for design purposes. The American Association of State Highway Officials (AASHO) has stated that the DHV represents the load that the highway must accommodate and largely determines the type of facility required and its characteristics (6). The DHV is selected in such a way that the highway under design should not experience extreme congestion at any time or unacceptable congestion for extended periods. However, the DHV must not be such that traffic would rarely be great enough to cause even minimal congestion because the facility would then be overdesigned and uneconomical.

Figure 7. Daily cost-volume relations at counter 36.

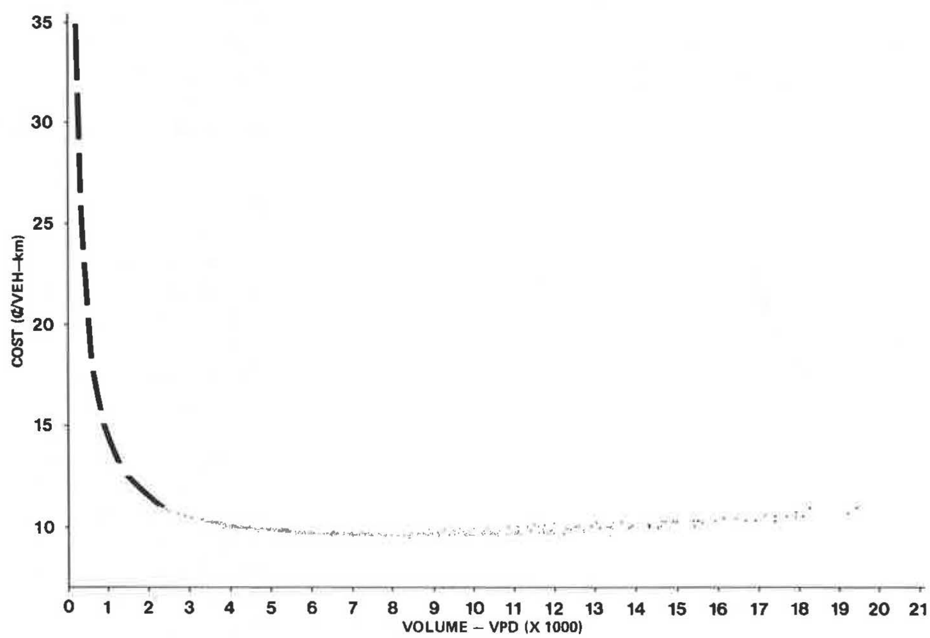
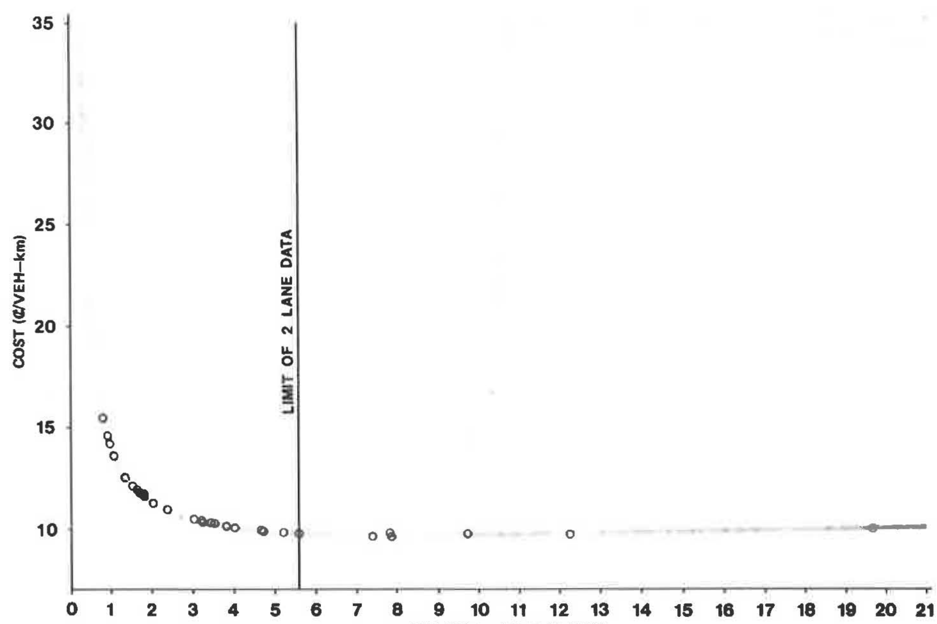


Figure 8. Annual cost-volume relations at Alberta permanent counters.



A common method of determining the DHV, proposed in the 1950 Highway Capacity Manual (7), involves the use of a graph that shows the highest hourly traffic volumes of the year according to rank. The 30th highest hourly volume is used by a number of agencies as the DHV for rural highways on the basis that the slope of the curve changes rapidly at that point and it is there that the ratio of benefits to expenditures is near the maximum. In a

Figure 9. Curves for 5500 highest hours of the year at four Alberta counter locations.

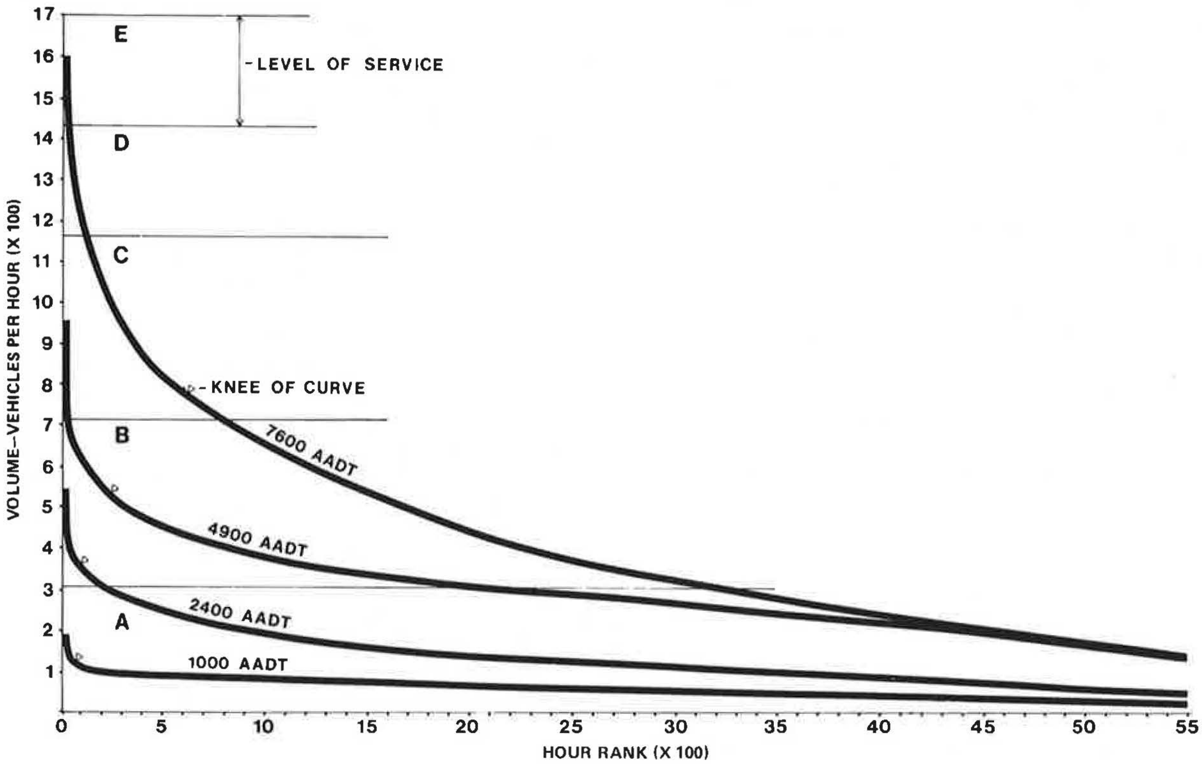


Table 1. K-factors for 30th highest hour of the year and knee of the curve.

Counter	Location	Type of Facility	AADT	Year	Hour at Knee	K-Factor: Knee	K-Factor: 30th Highest Hour
78	Highway 14	Two-lane undivided	1 000	1976	50	13.5	14.5
111	Highway 16	Two-lane undivided	2 400	1976	100	14.2	16.2
63	Highway 11	Two-lane undivided	4 900	1976	260	10.8	14.8
63	Highway 11	Two-lane undivided	5 600	1977	290	10.0	14.2
36	Trans-Canada Highway	Four-lane divided	7 600	1976	600	10.2	18.1
36	Trans-Canada Highway	Four-lane divided	7 800	1977	620	10.5	17.6
60	Highway 2	Four-lane divided	9 750	1977	570	9.7	17.0
57	Highway 2	Four-lane divided	12 250	1977	440	9.6	14.0
102	Highway 16	Four-lane divided	19 700	1977	480	10.1	12.7

Table 2. Tentative design standards for rural highways in Alberta.

Design Classification[a]	Type of Highway	No. of Lanes	Surface Width[b] (m)	Maximum Posted Speed (km/h)	20-Year Design AADT	Level of Service	DHV K-Factor
RFD-4[c]	Rural freeway divided	4		110	30 000-60 000	B-C	0.10
RAD-4[c]	Rural arterial divided						
RAU-213[d]	Rural arterial undivided	2	13	100	6000-10 000	B-C	0.10
RAU-211[d]	Rural arterial undivided	2	11	100	5000-9000	B-C	0.11
RAU-209[d]	Rural arterial undivided	2	9	90	4000-8000	B-C	0.12
RCU-209[d]	Rural collector undivided						
RCU-208[d]	Rural collector undivided	2	8	90	3000-7000	B-C	0.13
RLU-208[d]	Rural local undivided						

[a] Alberta Transportation classifications.
[b] Outside shoulder to outside shoulder.
[c] Design volume varies because of traffic composition and directional split.
[d] Design volume varies because of traffic composition and passing sight distance.

case in which the slope changes rapidly at some point other than the 30th highest hour, the DHV is chosen at the "knee of the curve".

No apparent attempt was made by the proposers of the method discussed here to justify or prove that these points in the curve do in fact provide the greatest economic benefit. Further, there is no clear indication as to what level of service should be chosen for the DHV. This has been left to the agency to determine and usually has been a policy decision of one form or another that is often made on a very obscure basis.

It is standard practice to determine a future DHV by multiplying the estimated or forecast AADT by a value K (i.e., the ratio of DHV to AADT), the hour used often being the 30th highest hour that is expected to occur in some future design year.

The 1965 HCM (4) recognizes the problem of selecting a measured or predicted traffic volume to be used for design purposes and, although the 30th highest hour is discussed, the HCM states the following: "This frequent reference to the 30th highest hour should not be misconstrued as a recommendation for rigid adoption, but rather as an example of typical highest hour relationship and trends." The following discussion is intended to provide some further insight into the process of selecting the DHV K-factor.

Highest-Hour-of-the-Year Signatures and K-Factors

It appears that highway agencies have traditionally ranked only the first 100 to 250 highest hours of the year. The remainder have been considered of little importance because the knee of the curve was usually evident within the first 100 hours. Based on a limited sample of Alberta counter locations, where all 8760 h were ranked, it appears that this is not the case. The highest 5500 h of four of these locations are shown in the graph in Figure 9. The knee of the curve is very evident for the lower-volume road; however, as the AADT increases, the knee disappears from within the first 100 h and shifts to somewhere in the 200- to 600-h range. This shift, of course, results in different K-factor values for the knee of the curve than for the 30th highest hour. Table 1 compares K-factors based on the hour at the knee of the curve with those based on the 30th highest hour. The knee-of-the-curve values are lower, within a narrower range, and tend to decrease in value as AADT increases. There is also a tendency for the knee of the curve to occur at a higher-ranked hour as AADT increases.

Formulation of Design Criteria

In arriving at a basis for choosing a DHV and a level of service for the DHV, the following guidelines are proposed:

1. The DHV chosen for the highway under design should be such that traffic demand for other higher hours of the year will not exceed the capacity of the facility for even short intervals of time except under rare or very exceptional circumstances.
2. The level of service chosen should provide the driver with various degrees of choice of speed and freedom from tension consistent with the length, duration, and purpose of the trip.
3. The attitude of motorists toward adverse operating conditions is influenced by their awareness of the environment in which they are traveling (e.g., difficult topography and built-up areas) and their recognition of associated practical cost limitations that preclude the design of the ideal facility.

The computations presented in this paper are based on a limited analysis that has given some further insight into formulating design criteria. Based on the rationale that the most economical DHV occurs where the slope of the curve for the highest hours of the year changes most rapidly, the knee of the curve and associated K-values appear to be most appropriate even though there is no known quantitative basis for their use. It follows that the level of service for the DHV should be equivalent to the V/C ratio where total unit costs are minimized.

Although no clear mandate has been presented, the approach suggested here will permit planners and designers to develop and select criteria on a more sound economic basis. This, of course, results in a wide range of DHVs and levels of service. These are given in Table 2, which has been formulated based on the work presented here. Since the table represents a very limited number of site-specific cases, it is by no means final.

CONCLUSIONS AND RECOMMENDATIONS

Although this analysis has been rather limited in scope because of the lack of data and the use of manual methods, we feel that the work is sufficiently valid to make some preliminary conclusions and recommendations for further research. The following conclusions can be made:

1. A cost-effective V/C ratio can be computed for various supply conditions by using the technique described.
2. Evidence presented on the demand side further supports the use of K-values for the knee of the curve rather than use of the 30th highest hour for the DHV because the values are more consistent.
3. Daily and yearly cost-volume relations do not indicate a cost-effective volume as clearly as does the hourly measure.
4. Economic justification for converting existing two-lane facilities into multilane facilities (as the public now demands) does not appear evident based on the measures of operating efficiency presented. However, the technique is felt to hold some merit as one of the parameters for priority rating.

The following recommendations are made for further research:

1. Procedures for measuring economic operating efficiency should be refined. Several areas require attention, namely (a) capacity and level-of-service volumes and corresponding speeds for two-lane roads (speed-volume curves) require validation (this is currently one of the greatest gaps in two-lane highway capacity theory), (b) vehicle operating costs for Alberta should be updated, and (c) value of time requires considerably more analysis and understanding and the derivation of values for different trip purposes and trip lengths.
2. Data for two-lane roads with higher AADTs should be analyzed to validate further the concept of the knee of the curve for DHV, the K-values derived so far, and the

unit average cost supply functions (Ontario may be one of the few Canadian sources for this information).

3. Although direct relations between the knee of the curve and the K-factor and cost-effective V/C ratios can be shown, the relation between economic level of service (supply) and DHV (demand) is still obscure and requires further research.

ACKNOWLEDGMENT

The observations and views presented in this paper are strictly our own. The design criteria presented are not formal Alberta Transportation policy.

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Freeway Level of Service: A Revised Approach

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Concepts, philosophies, and standards for freeway level of service presented in the 1965 Highway Capacity Manual are reviewed. A revised approach is developed that incorporates density in the definition of standards. Speed-flow relations under ideal conditions are approximated based on secondary source data and a limited number of pilot field surveys associated with current work. The recommendations made for new level-of-service standards for freeways are based on recalibrated speed-flow relations and incorporate density as a parameter.

The basis for any technique of capacity analysis is the definition of quality-of-service criteria and the correlation of these criteria with operational and design parameters. The 1950 Highway Capacity Manual (1) defined service in terms of "possible" and "practical" capacity. Practical capacity represented the maximum traffic volume that could be accommodated (under prevailing roadway and traffic conditions) while an acceptable quality of service was provided.

The 1965 Highway Capacity Manual (HCM) (2) introduced the concept of level of service, which allows for a more detailed treatment of service quality. The 1965 HCM defines level of service as "a qualitative measure of the effect of a number of factors, which include speed and travel time, traffic interruptions, freedom to maneuver, safety, driving comfort and operating cost" on operations. It also defines six levels of service—A through F—which describe a wide range of conditions, from totally free at level A to forced flow at level F.

CURRENT STANDARDS FOR LEVEL OF SERVICE

Current standards for freeway level of service are given in Table 9.1 of the 1965 HCM (2, pp. 252-253). Each level is a range of operating conditions for which the table defines boundary conditions in terms of two parameters: (a) volume-to-capacity (V/C) ratio, which may be stated as a volume, and (b) operating speed. Table 9.1 gives minimum values of operating speed and maximum V/C values for each level of service. The standards in the table apply under "ideal" conditions, which include (a) no trucks or buses in the traffic stream, (b) 3.6-m (12-ft) minimum lane widths, and (c) no obstructions in the median or roadside area closer than 1.8 m (6 ft) to the pavement edge. The standards for the V/C ratio depend on average highway speed, which is a weighted average design speed for the highway segment under study.

For a highway segment to be said to operate under a particular level of service, the criteria for both V/C ratio and operating speed must be met. This is an important point. The standards in Table 9.1 of the 1965 HCM do not, nor were they intended to, represent a correlation between speed and V/C ratio. The existence of a V/C ratio appropriate for level of service C does not guarantee that the operating speed for that level will also be met. This characteristic of the standards leads to a number of problems in their use.

QUESTIONS, ISSUES, AND ALTERNATIVES

In formulating recommendations for level-of-service standards, a number of critical philosophic and practical issues must be raised. The resulting recommendations should meet two primary objectives:

1. Levels of service must be defined in terms that are meaningful for the driver who experiences them and meaningful to the planners, analysts, and designers who will use the standard.
2. Definitions of level of service must be consistent with each other and consistent in application to the various types of subsections that occur on a freeway (i.e., open sections, weaving areas, and ramp terminals).

A number of key issues concerning the concept of level of service are treated here in the context of these general objectives along with practical questions about the state of the art and the availability of data.

Continued Use of the Level-of-Service Concept

The first major question is, Should the use of the basic concept of levels of service as quality descriptors be continued?

Essentially, there are only two alternatives to the concept of level of service: (a) a structure of capacity of the type found in the 1950 HCM (1) followed by design levels that represent "adequate" service quality or (b) a treatment of speed-volume relations as continuous functions. The first alternative is clearly a step backward and is really a level-of-service concept itself, modified by having only three levels. Such a structure might indeed be adequate in design [the American Association of State Highway and Transportation Officials (AASHTO) already specifies level of service as part of its design criteria] but would severely limit the use of procedures in analysis, where greater detail is needed.

The second alternative suggests a radical change and requires extensive calibration of generalized speed-volume curves. Although considerable speed-flow data are available, base conditions—e.g., percentage of trucks, number of lanes, and design speed—vary widely, which makes calibration of a full generalized speed-flow curve or curves difficult. In any event, where design is considered, standard or threshold levels would have to be established.

The two alternatives outlined are really extremes of a similar concept. The essential issue is how many threshold levels will be identified. In the 1950 HCM, the answer was three: possible, practical-rural, and practical-urban capacity. For the second alternative, the answer was infinite: a continuous relationship. Levels of service as they now stand define five boundary conditions in specific terms and a sixth to describe the entire range of unstable or forced flow.

An even more exotic alternative does exist. One might attempt to index level of service to various microscopic physiological parameters concerning driver experience and behavior. Studies have been made that relate such parameters as steering wheel reversals, heart rates, and blood pressure to traffic conditions. Such measures, however, although interesting, are not highly useful to designers, planners, and analysts who must deal in standard parameters of traffic flow and highway design. The state of the art in this area does not permit consistent correlation between standard flow measures and physiological factors.

Another approach would be to tie level of service to overall door-to-door trip convenience. This would permit multimodal evaluations but, again, the state of the art is insufficient to allow serious consideration of this option.

There are no compelling reasons to reduce or increase the number of levels defined. In the weaving procedure developed as part of project 3-15 of the National Cooperative Highway Research Program (NCHRP) (3), the researchers did divide level of service D into two sublevels called D1 and D2. This was done because an observed breadth of conditions occurred in level D. There is no evidence, however, to suggest that level D should be split for all freeway cases and, indeed, the observed breadth observed may have been an accident of calibration.

On the other hand, there are a number of compelling arguments for retaining the level-of-service concept in its current form:

1. The concept is now a familiar one that can be readily used and understood by professionals and many technicians in the field;
2. Many associated government standards, such as AASHTO design standards and recent government standards on traffic noise, are formulated in terms of level of service; and
3. Most of the extant material on freeway capacity analysis developed since the 1965 HCM was drafted is also based on level of service.

The level-of-service concept is a viable mechanism for describing service quality for freeways that is strongly established in the profession. Although the ultimate fate of level of service as a concept must await the results of other research in other areas of capacity, we endorse its use in the context of this work.

Table 9.1: Defined Standards or a Relationship

The V/C and speed specifications for freeway levels of service found in Table 9.1 in the 1965 HCM (2, pp. 252-253) are separately defined. Volumes and speeds in that table are not intended to be, nor are they in fact, correlated.

Chapter 5 of the 1965 HCM contains typical speed-flow curves from an early study by the U.S. Bureau of Public Roads. Our Table 1 compares volumes from Table 9.1 of the 1965 HCM and those depicted by the speed-flow curves shown in Figure 3.38 in that volume (2, p. 62).

Similar comparisons may be drawn for average highway speeds of 97 and 80 km/h (60 and 50 mph). Note that for levels A and B the standards given in Table 9.1 of the HCM agree closely with values taken from the speed-flow curves. At level C, volumes given in Table 9.1 appear to be higher than those from the curves. Level C volumes from Figure 3.38 do, however, agree closely with values in Table 9.1 for peak-hour factor (PHF) $\cong$ 0.95 (interpolating). Since the base conditions for Figure 3.38 include a "high PHF, approximating 1.00", a value of 0.95 is probably close to what the data represented.

The major discrepancy between Table 9.1 and Figure 3.38 occurs consistently at level of service D. At this level (for all values of average highway speed), volumes in Table 9.1 are considerably lower than those indicated by the speed-flow curves—lower by as much as 520

Table 1. Comparison of HCM volume levels for various operating speeds.

		Volume[a]					
		Four Lanes		Six Lanes		Eight Lanes	
Level of Service	Operating Speed (km/h)	Table 9.1	Figure 3.38	Table 9.1	Figure 3.38	Table 9.1	Figure 3.38
A	96	1400	1390	2400	2340	3400	3440
B	88	2000	2080	3500	3510	5000	5000
C	80	3000	2790	4800	4500	6600	6280
D	64	3600	3860	5400	5790	7200	7720
E	48	4000	4000	6000	6000	8000	8000
F	-	-	-	-	-	-	-

Notes: 1 km = 0.62 mile.
Average highway speed = 112 km/h.
[a]Peak-hour factor = 1.00.

vehicles/h (see Table 1). Note further that Figure 3.38 does not represent ideal conditions: The speed-flow curves include trucks and, in some instances, restrictive lane widths or lateral clearance or both. Thus, if they were corrected for ideal conditions (which is not possible from the data available), the volumes shown in Figure 3.38 of the HCM would be even higher, disrupting the apparent agreement at levels A-C and accentuating the discrepancy at level D.

More recent data seem to indicate that volumes for any given speed are even higher than those indicated by the HCM speed-flow curves. A study of the Southern State Parkway (4), for example, indicates the following volumes for the six-lane freeway (1 km = 0.62 mile):

Speed (km/h)	Volume (automobiles/h)	Speed (km/h)	Volume (automobiles/h)
48	6000	88	5500
64	5850	96	4950
80	5650		

These volumes are all considerably higher than those in Table 9.1 and Figure 3.38 of the HCM, strikingly so at the higher speed levels normally associated with levels A-C. The volume at 96 km/h (60 mph), for example, more than doubles those of either HCM source. The design speed of the Southern State Parkway is, moreover, 96 km/h, and the study was made well after the 88-km/h (55-mph) speed limit was in effect. In all other features, the Southern State Parkway is ideal: no trucks and good lane width and lateral clearance. The flows cited reflect a PHF of 1.00 but are based on 15-min intervals.

It appears that standards given in Table 9.1 in the HCM show volume levels that are below those that will actually occur at the operating speeds indicated, perhaps seriously so. The corollary to this is that, for the volumes shown in Table 9.1, operating speeds will be higher in practice than those shown.

Remember that Table 9.1 was not intended to reflect a speed-flow relation. The issue is clear: Should it reflect a relationship, or should both speed and V/C standards be defined?

The arguments for adopting a relationship base for Table 9.1 are strong:

1. In use, Table 9.1 often demands the assumption that speed and V/C are correlated. Designers select a level of service and design for it by using only V/C. They must presume that the indicated speeds will result. Many analyses are done without field measurements of speed, and level of service is determined, again, by V/C alone. In use, V/C is clearly the primary measure; most often the operating speed is assumed to follow.

2. If volumes in Table 9.1 are consistently lower than those that regularly occur in the field for the speeds shown, the implication is that V/C alone will determine level of service because the operating speed limits would never be the controlling factor. The two-parameter standard becomes a fiction since only one is ever effective. The fact that V/C is really the effective standard in Table 9.1 is, however, consistent with the use of the table, in which V/C is often the only value used.

3. The use of freeway capacity procedures in analysis is considerably hampered and restricted if the speeds and volumes in Table 9.1 do not correlate. No analysis could be properly done without data on operating speed, and such data are difficult to measure in the field and are far less available than volume data, which are more routinely collected.

We strongly believe that Table 9.1 should represent correlated values of V/C and speed and that such a relationship should be calibrated to the extent possible.

Speed Criteria

The 1965 HCM defines speed criteria for freeways in terms of operating speed, which is defined as "the maximum safe speed for given traffic conditions that an individual vehicle can travel at if the driver so desires, without exceeding the design speed at any point" (2, p. 246). Two alternatives to operating speed may be considered: (a) average running speed (space mean speed) and (b) percentile speeds (e.g., 85th percentile speed).

Operating speed is a difficult parameter to work with, particularly when speed-flow relations are to be calibrated. It is, most properly, a parameter measured by using runs by a test automobile (by the "maximum car technique") and is not a statistic that can be isolated from sample measurements of the traffic stream. When test-run measurements do not exist, operating speed can only be roughly estimated. Even when test-run results are available, they may vary considerably depending on the driver. Rarely are sufficient test runs made to statistically dampen this factor.

Average running speed is a statistical parameter that may be computed from sample observations of the traffic stream. Its relation to operating speed varies, but it is generally from 4.8 to 8 km/h (3 to 5 mph) lower at high levels of service and almost equal at capacity. Average running speed is more universally understood than operating speed, which is subject to frequent misinterpretation and is the standard used by AASHTO. Use of a stream statistic enables sample data to be used in calibrating relationships. It is also interesting to note that, in the basic traffic-flow relation,

$$\text{Volume (vehicles/h)} = \text{density (vehicles/km)} \times \text{speed (km/h)} \quad (1)$$

space mean speed (a statistical term for average running speed) is the parameter that must be used.

The use of an average running speed does present one philosophic problem. Table 9.1 in the HCM depicts speeds for ideal conditions—in particular, no trucks. Through the use of truck equivalencies and truck factors, service volumes for prevailing conditions are computed for various levels of service. Suppose, for example, that level of service C has a service volume of 2500 automobiles/h under ideal conditions for a given highway and a corresponding average running speed of 80 km/h (50 mph) (threshold values). It is further determined, by using truck factors, that a service volume of 2000 vehicles/h yields the same level of service. Is the corresponding speed still 80 km/h?

If the calibration of truck factors were based on finding volume levels that produce equivalent speeds, the answer would be yes. But none of the available methods for computing truck factors do this. As a result, the answer is no: The speed would probably be lower because of the percentage of trucks, which generally travel slower than automobiles. Thus, there is, at least on a philosophic level, a question as to the real meaning of the speed values in a recalibrated table in which average speed is used.

The 85th percentile speed is an intriguing alternative.

It is a statistic that can be isolated from stream data and, as a higher percentile (a positional value), may be expected to be relatively stable throughout the normal range of truck percentages. Operating speed also possesses this characteristic and would not vary widely with the presence of trucks at equivalent volumes.

All things being equal, we would recommend the use of the 85th percentile speed as a standard because it is (a) a stream data statistic, and (b) relatively insensitive to the presence of trucks at equivalent volumes.

Unfortunately, most extant data for calibrating speed-flow relations do not allow the 85th percentile speed to be isolated. To find the 85th percentile speed, individual vehicle speed would have to be recovered, or a computation of the standard deviation would have to be available. Because of this, it is recommended that average running speed be used as a criterion despite its sensitivity to the presence of trucks. The degree of sensitivity is not really known and might potentially be smaller than the normal spread of speed-flow data. Three overriding concerns dictate this choice (two of them have already been discussed):

1. A traffic stream statistic should be used to define levels of service to allow extant data to be used.
2. Average running speed is the proper statistic to use in speed-flow-density relations.
3. The NCHRP project 3-15 procedure for weaving areas on freeways also used average running speed.

The third point is important in terms of the consistency of procedures developed for freeways in current work.

Peak-Hour Factor

Table 9.1 incorporates the use of PHF at levels C and D. The meaning of the use of PHF in the HCM is not clear to many users and is confusing in that levels of service are defined for a peak 5-min period but are applied over a full hour during which flow may vary considerably.

Essentially, the HCM use of PHF allows that design and analysis at levels C and D are based on the peak 5-min rate of flow during the hour of interest (usually the peak). PHF is not used at other levels for two reasons:

1. At levels of service A and B, peaking within the hour will merely reduce the service provided for short periods and not cause any congestion or traffic backup.
2. At level of service E (capacity), PHF is 1.00 by definition.

The second point might be disputed. A PHF of 1.00 is never observed in the field whereas values of 0.95-0.97 are. Further, volumes of 2000 automobiles/h/lane have been observed at such PHFs. It is also true that a facility may reach capacity for a period of time less than 1 h, and according to the HCM this is not clearly identified.

The interpretation of Table 9.1 in the HCM for levels C and D is also unclear. Are the speeds given also for the peak 5-min period or for the whole hour? If the former, how can the hour as a whole be described? If the latter, then the same speed is associated with widely variant flow levels and distributions.

The difficulty is that few hours experience uniform operating conditions, even the peak hour. Operating conditions may vary by several levels within an hour. Consider the following situation: 2800 automobiles/h, a four-lane freeway, ideal conditions, and PHF = 0.77. According to Table 9.1, this is in level of service D for the full hour. Actually, during the peak 5 min, a flow rate of 3600 automobiles/h is experienced (level D). For the rest of the hour, the average flow rate is given by

$$[2800 - (3600/12)]/(11/12) = 2727 \text{ automobiles/h} \tag{2}$$

Taken as an average, this is in level C. Obviously, this volume too will vary from period to period, but the point is clear: What Table 9.1 in the HCM labels as level of service D for an hour may be level C or better for a good portion of that hour. Perhaps Table 9.1 should not be geared to describing a full hour of operation but rather some shorter, reasonably stable period of time.

Were Table 9.1 to be based on peak flow rates (they do not actually have to be peak but simply uniform flow rates), the consideration of PHF could be greatly simplified. It would not appear as a factor in the table at all, and users would be instructed to enter the table with the volumes adjusted to peak flow rates by means of the following:

$$\text{Peak flow rate} = \text{volume/PHF} \tag{3}$$

This implies that PHF will be considered at all levels of service. At levels A and E, this is of little importance. At level E, it will permit proper accounting for situations in which capacity is experienced for a portion of an hour and better levels exist during other portions of the hour. At level A, short periods of free flow may be identified even if other portions of the hour operate at poorer levels.

Level of service B, however, is used as a design standard for rural highways. Currently, level B does not consider a peak flow. However, once both the criteria for the standard and its use are adjusted to include a peak flow rate, the effect on design would not be significant because of this factor. It should be noted that many, if not all, designs based on a recalibrated Table 9.1 would be affected to some degree simply because of the calibration of new numeric limits at each level.

It is recommended that the recalibrated Table 9.1 in the HCM be based on peak flow rates and that, before the table is entered, the PHF expansion be applied directly to demand volumes.

RECOMMENDATIONS

As a result of these and other considerations, it is recommended that the development of recalibrated level-of-service standards for freeways be based on the following:

1. Table 9.1 should be representative of speed-flow relations and should be calibrated by using the best available data.
2. Average running speed should be used to establish speed criteria for the various levels of service.
3. Table 9.1 should be recalibrated by using a base of peak flow rates.
4. The effect of the 88-km/h (55-mph) speed limit must be accounted for in recalibrating Table 9.1, but there is no compelling reason to avoid showing speeds equal to or higher than 88 km/h in the standards.

The last point follows from the use of speed-flow correlations as the basis for the calibration of standards. If such calibrations showed speeds higher than 88 km/h, they would have to be accepted.

RECALIBRATION OF LEVEL-OF-SERVICE STANDARDS

To recalibrate level-of-service standards based on speed-flow relations, it is necessary to acquire a data base that consists of measured speeds and volumes under controlled conditions. Such data are sparse in the literature, particularly with regard to "controlled conditions". To calibrate speed-flow relations properly, underlying conditions must at least be known if not uniform. These underlying conditions include (a) the presence of trucks, (b) lane widths and lateral clearance, (c) the period of time over which flows are measured, and (d) average highway speed.

Where matched speed-flow data do exist in the literature, these underlying conditions are generally not specified; where they are specified, they vary considerably from study to study. Useful data were obtained from a relatively small number of sources:

1. The 1965 Highway Capacity Manual—The HCM shows typical speed-flow curves for a variety of freeway types, including stratifications by four-, six-, and eight-lane freeways; average highway speeds of 112, 96, and 80 km/h (70, 60, and 50 mph); the effect of speed limits; and the use of either operating or average running speed as a parameter.
2. Consultant studies—Data from the Southern State

Figure 1. Results of field survey.

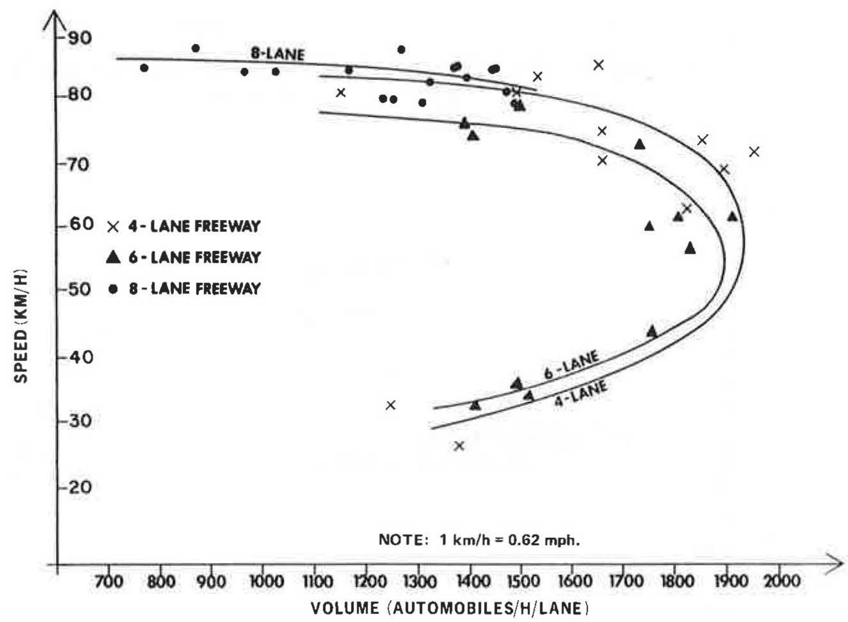


Figure 2. Speed-flow data for eight-lane freeways at 112 km/h.

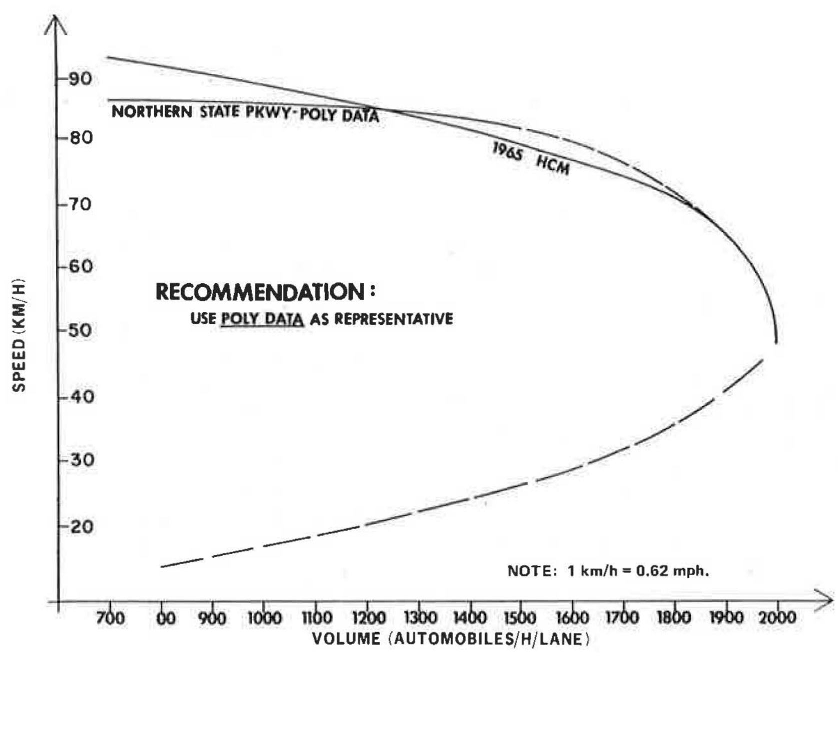


Figure 3. Speed-flow data for six-lane freeways at 112 km/h.

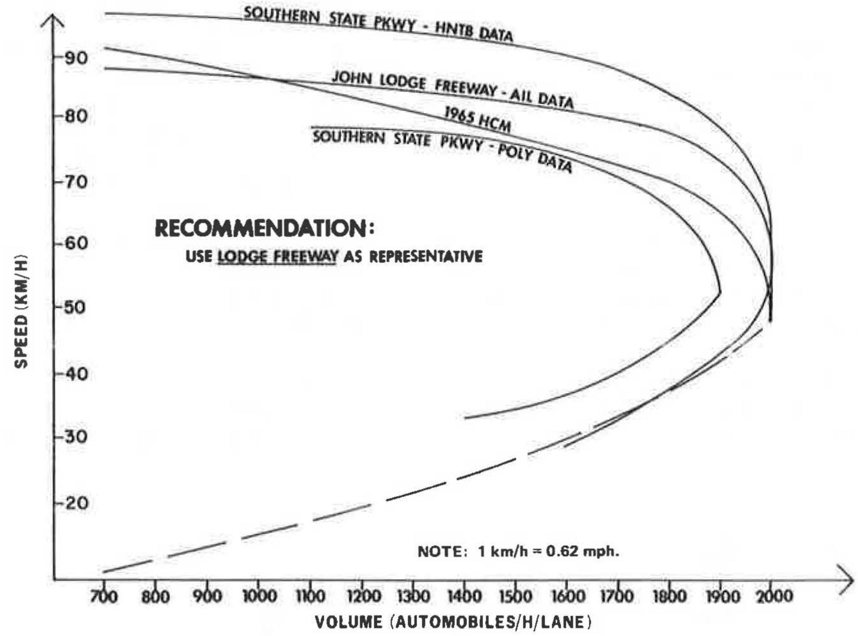
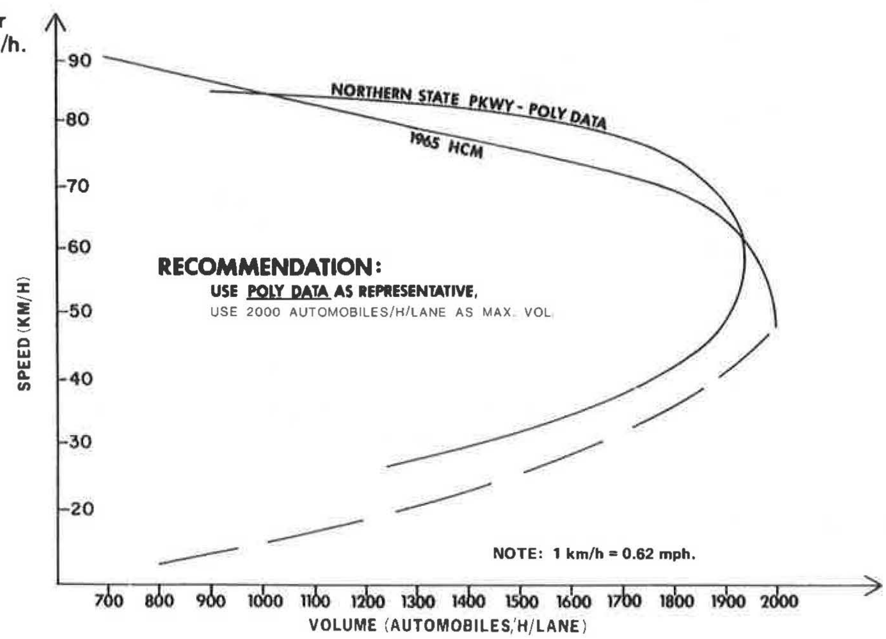


Figure 4. Speed-flow data for four-lane freeways at 112 km/h.



Parkway in New York (4) and the Lunalillo Freeway in Hawaii have proved to be most useful. A study of traffic flow models conducted by Airborne Instruments Laboratory (5) contains useful speed-flow data from the John C. Lodge Freeway in Detroit.

It had originally been thought that the several operational surveillance systems in the United States would be excellent sources of speed-flow data. Actually, few such systems even measure speed but rather use occupancy as a principal parameter. Where speed is observed, it is usually not at the same point for which volumes are available. Further, retrieval of surveillance system data in useful form is in itself a major effort that entails considerable expense.

Because of the small number of extant data sets that can be used in the establishment of general speed-flow curves, three field surveys were done on parkways in the New York area. These facilities come closest to providing truly ideal conditions—i.e., no trucks or buses, 3.6-m (12-ft) lane widths, and adequate lateral clearances. The results of these field surveys are shown in Figure 1. One survey each was conducted on a four-lane, a six-lane, and an eight-lane section of freeway.

Figures 2 through 6 show all available data and speed-flow relations stratified by type of freeway. In formulating recommendations for representative "general" curves, the HCM data were given the least weight because of their age.

Figures 2 through 6 illustrate a number of interesting points:

1. There are not enough data to suggest whether or not

Figure 5. Speed-flow data for 96-km/h freeways.

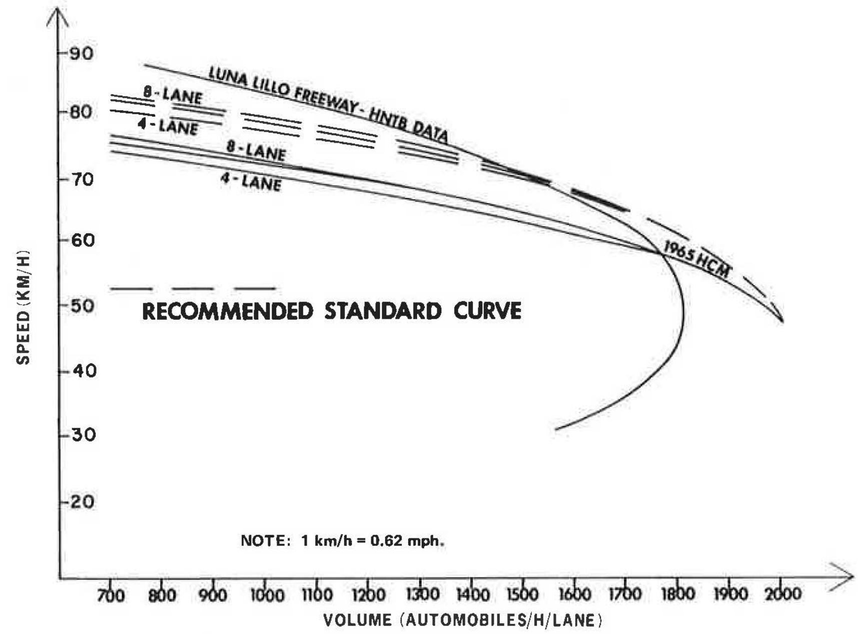
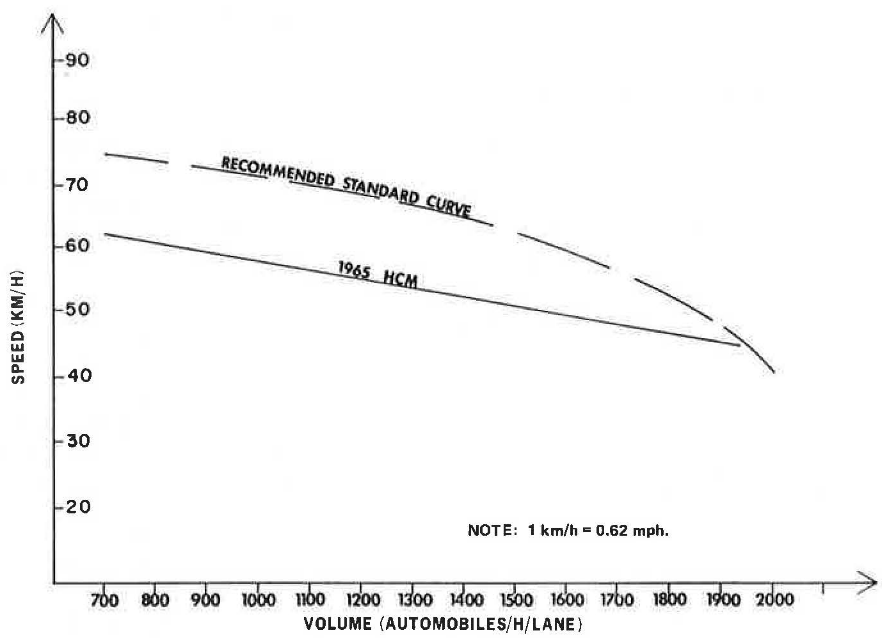


Figure 6. Speed-flow data for 80-km/h freeways.



2000 automobiles/h/lane is an appropriate value for maximum capacity under ideal conditions. Although none of our field studies reached that level, other sources did. Thus, there is no reason to either increase or decrease the 2000-automobiles/h/lane maximum at this time.

2. All of the more recent data show a wide range of volumes for which speed is relatively constant. This is not indicated in the HCM curves and will have to be dealt with in terms of level-of-service standards.

3. As a corollary to item 2 above, all of the more recent studies show a rapid deterioration of speed over a small range of volumes as the level of 2000 automobiles/h/lane is approached. This, too, has drastic consequences in the definition and interpretation of level-of-service standards.

On each of the curves, a general recommendation is made concerning the shape of a standard speed-flow curve. In Figure 6, the recommended curve for an average highway speed of 80 km/h (50 mph) is merely an extrapolation of the trends observed in other figures since there were no data available for this case.

Figure 7 shows the recommended standard curves for use in developing level-of-service standards. Because of the paucity of data, these curves are not adequately calibrated in the statistical sense but are "eyeball fits to the available data". But we believe that they are far more representative of current traffic characteristics than the curves that appear in the 1965 HCM.

Level-of-Service Standards Defined by Speeds

The most straightforward approach to defining levels of

Figure 7. Recommended standard speed-flow curves.

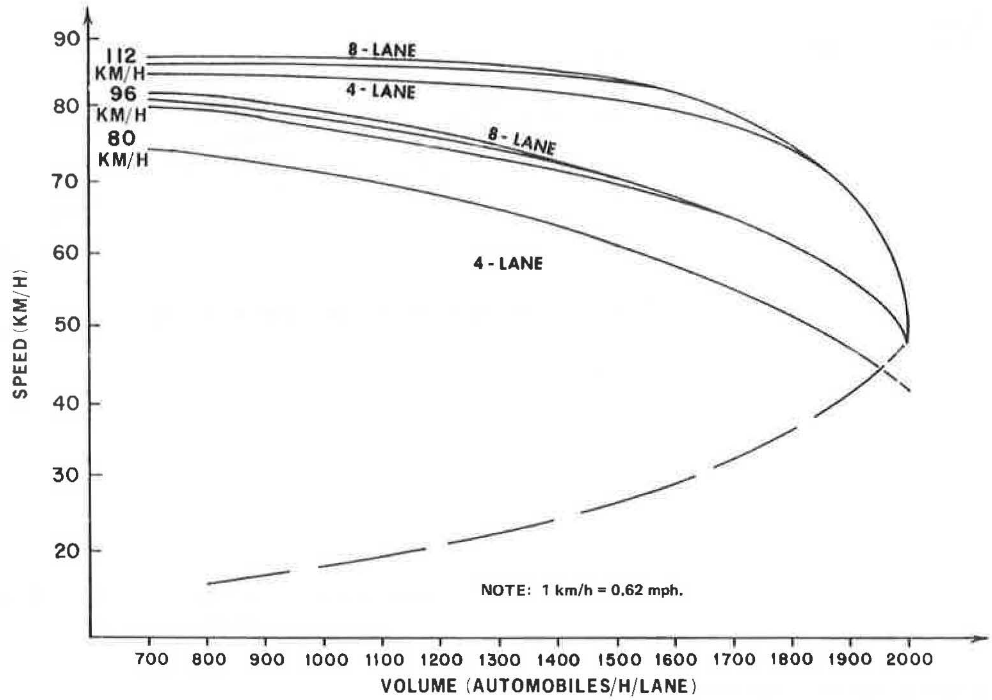


Table 2. Level-of-service standards defined by speed.

Avg Highway Speed (km/h)	Level of Service	Avg Running Speed	Maximum Service Volume for PHF = 1.00 Four Lanes	Six Lanes	Eight Lanes	Each Additional Lane
112	A	96	3120	4980	6640	1660
	B	88	4680	5520	7360	1840
	C	80	3880	5820	7760	1940
	D	64	3970	5955	7940	1985
	E	48	4000	6000	8000	2000
	F	48	-	-	-	-
96	A	96	1400	2520	3680	940
	B	88	2680	4470	5960	1490
	C	80	3420	5130	6840	1710
	D	64	3800	5700	7600	1900
	E	48	4000	6000	8000	2000
	F	48	-	-	-	-
80	A	96	-	-	-	-
	B	88	1840	2760	3680	920
	C	80	2780	4170	5560	1390
	D	64	3360	5040	6720	1680
	E	48	4000	6000	8000	2000
	F	48	-	-	-	-

Note: 1 km = 0.62 mile.

service would be to define a speed range for each. Thus, service would be defined in terms meaningful to the user and would be correlated with volumes that may actually be anticipated. Since none of the speed-flow curves reach more than 83 to 85 km/h (52 to 53 mph), the most logical definitions would be those given in the table below (1 km = 0.62 mile):

Level of Service	Speed (km/h)	Level of Service	Speed (km/h)
A	≥80	D	≥56
B	≥72	E	≥48
C	≥64	F	<48

Table 2 gives the level-of-service standards that result from these definitions. Volumes are taken from Figure 7.

Note that the format of Table 2 follows previous recommendations. Volumes are shown only for a theoretic PHF of 1.00; that is, peak flow rates are shown. Figures 1 through 7 are also based on flow rates; the time period varies from 2 min for the John C. Lodge Freeway to 15 min for the Lunalillo Freeway and the Southern State Parkway (4).

Because of the peculiar characteristics of the recommended standard curves—i.e., a wide range of volume with constant speed followed by a rapid deterioration of speed as volume approaches 2000 automobiles/h/lane—the entire range of levels of service only covers a relatively small range of volumes. For average highway speed of 112 km/h (70 mph), this range is approximately 1660 to 2000 automobiles/h/lane, a range that is almost entirely within level of service E by current standards.

These standards, then, are not really useful to the designer, who could not reasonably design anywhere in the available range in most cases. Nor are they particularly useful to the analyst since they do not contain any description of what could reasonably be called free flow or anything approaching it. For these reasons, levels of service based on speed alone are not recommended.

A Philosophy of Level of Service

The three parameters that describe the state of a traffic stream are speed, volume (or flow), and density. Level-of-service standards are generally based on speed and volume because these parameters are easily observed and measured in the field. Density, which is difficult to measure directly and often must be measured by using aerial photography, can be computed from speed-volume data.

A level of service is a measure of quality that is intended to describe the quality of service being provided to the motorists who use a facility. The many parameters that affect the driver's perception of quality of service are all related to the ease and comfort with which the driver is able to proceed. In terms of the major parameters of

Table 3. Levels of service for basic freeway segments.

Avg Highway Speed (km/h)	Level of Service	Speed (km/h)	Density (automobiles/ km/lane)	Expected Service Volume[a] (automobiles/h)			
				Four Lanes	Six Lanes	Eight Lanes	Each Additional Lane
112	A	≥80	≤9	1600	2400	3280	820
	B	≥80	≤16	2500	3900	5400	1350
	C	≥77	≤22	3400	5100	6800	1700
	D	≥64	≤29	3850	5775	7700	1925
	E	≥48	≤42	4000	6000	8000	2000
	F	<48	-	-	-	-	-
96	A	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]
	B	≥72	≤16	2300	3525	4800	1200
	C	≥69	≤22	3050	4575	6100	1525
	D	≥67	≤29	3600	5400	7200	1800
	E	≥48	≤42	4000	6000	8000	2000
	F	<48	-	-	-	-	-
80	A	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]
	B	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]
	C	≥64	≤22	2800	4200	5600	1400
	D	≥56	≤29	3300	4950	4950	1650
	E	≥48	≤42	4000	6000	6000	2000
	F	<48	-	-	-	-	-

Note: 1 km = 0.62 mile.

[a]One direction, for levels of service during uniform periods of traffic flow.
[b]Level of service not achievable because of restricted average highway speed.

Table 4. V/C values for use in design.

Avg Highway Speed (km/h)	V/C Ratio	Avg Running Speed (km/h)			Density (automobiles/ km/lane)			Level of Service		
		Four Lanes	Six Lanes	Eight Lanes	Four Lanes	Six Lanes	Eight Lanes	Four Lanes	Six Lanes	Eight Lanes
112	0.20	83	86	86	5	5	5	A	A	A
	0.40	83	86	86	10	9	9	B	A	A
	0.60	83	84	86	14	14	14	B	B	B
	0.80	78	82	82	20	20	20	C	C	C
96	0.20	80	82	82	5	5	5	B	B	B
	0.40	78	80	80	10	10	10	B	B	B
	0.60	75	77	77	16	16	16	C	B	B
	0.80	67	69	69	24	23	23	C	C	D
80	0.20	75	75	75	5	5	5	C	C	C
	0.40	74	74	74	11	11	11	C	C	C
	0.60	69	69	69	17	17	17	C	C	C
	0.80	59	59	59	27	27	27	D	D	D

Note: 1 km = 0.62 mile.

stream flow, an interesting dichotomy develops:

1. The driver experiences (a) speed and (b) density (the relative proximity of other vehicles).
2. The designer or analyst is most interested in the volumes that can be accommodated.

Level of service should be defined in terms of the parameters directly experienced by drivers: speed and density. These should then be related to volumes for the use of designers, analysts, and planners.

Table 9.1 in the 1965 HCM is currently defined on the basis of constant speeds for each level of service. This leads to different V/C values for different highway types and markedly different densities, particularly for average highway speeds of 96 and 80 km/h (60 and 50 mph). For example, at 64 km/h (40 mph) (level D), density is approximately 28 vehicles/km/lane (45 vehicles/mile/lane) for an average highway speed of 112 km/h (70 mph); for an average speed of 80 km/h, density is 14 vehicles/km/lane (22.5 vehicles/mile/lane). Thus, two widely variant conditions of operation are labeled with the same level of service.

Of course, it is not possible to define both density and speed for a particular level because the two are related. The question, however, is whether or not defining level of service by speed alone, with no consideration of density, is proper or reasonable.

It is recommended that levels of service be established by considering both speed and density as defining parameters. Defining levels in this way considers both parameters of which drivers are directly aware (speed and density) and produces standards of a familiar form in a more meaningful way than does the current version of Table 9.1 in the HCM.

Recommended Standards for Freeway Level of Service

Table 3 gives the recommended standards for freeway level of service. They are in keeping with previous recommendations and have the following characteristics:

1. They are representative of observed speed-flow relations as shown in Figure 7.
2. They are based on average running speed as a speed parameter.
3. They are representative of peak flow rates, i.e., a PHF of 1.00.
4. Levels of service are defined by using both speed and density as parameters.

The principal defining parameter in Table 3 is density. Increments were chosen to be approximately representative of the six photographs in the 1965 HCM that illustrate the various levels of service. Speeds were established by using Figure 7. Since Figure 7 shows only volume and speed, the speed-volume point appropriate to a chosen density was determined by trial and error. Both the speed ranges and the density ranges given in Table 3 are approximate to within ± 2 units. This is reasonable in view of the approximate nature of the Figure 7 calibrations and the known spread exhibited by most speed-flow data.

Note that the V/C ratio is not given in Table 3. As long as 2000 automobiles/h/lane remains the accepted maximum capacity for all types of freeways, V/C and volume are directly related on a one-to-one basis. Since volume is the parameter of direct interest to designers, analysts, and planners, its direct use in the standards simplifies their use and interpretation. It should also be noted that, for clarity, the standards shown for average highway speeds of 112, 96, and 80 km/h (70, 60, and 50 mph) are all in the same format.

The recommended standards in Table 3 result in nonuniform ranges of volume for the various levels of service. Again, this is a result of observed speed-flow characteristics in which speed remains relatively constant over a wide range of volumes and then deteriorates rapidly over a relatively small volume range as 2000 automobiles/h/lane is approached. Because of this, these standards do not give the designer a great deal of flexibility. Design at levels of service C, D, or E on a freeway with 112-km/h (70-mph) average speed could not be attempted because all are in a fairly unstable range of flow and a small error in estimated volumes would mean regular breakdowns. This leaves just two choices for a design level of service: A or B.

Since two design levels may not give the designer enough flexibility to achieve designs of optimal efficiency, it is recommended that a corollary table be developed for their use. Table 4 indicates, for uniform increments in V/C ratio of 0.20, the average running speed and level of service that could be expected if design at such a V/C value were attempted. In this table, the designer is presented with a wider range of feasible design levels.

IMPLICATIONS OF REVISED STANDARDS

Should the recommendations made here be adopted for freeway level-of-service standards, the manner in which such standards are used and interpreted would change even though their final form is very similar to standards in the current HCM. The standards given in Table 3 show speed ranges for the various levels of service that are not exclusive; i.e., several levels may have the same speed range. A field determination of level of service will require a determination of both speed (average running) and density. This, however, is no more complicated than current standards that require both speed and volume for such determinations. Density would not be observed directly but would be computed from speed and volume.

Further, it is hoped that the standards given in Table 3 are reasonably representative of what generally happens in the field under ideal conditions. Thus, where only volume data are available, it may be assumed that the speed shown in Figure 7 and the resultant density are in the range of what would be expected in the field. This statement could be considerably strengthened if the data base for Figure 7 were stronger. Clearly, more studies in this area, as well as closer control of underlying variables, are called for.

Finally, the traditional use of levels of service C and D for urban design would be altered because both are too close to the 2000-automobiles/h/lane mark for reasonable stability. A and B might be used as design levels or intermediate V/C points as indicated in Table 4. It is, however, clear that the design levels of service specified in AASHTO and other documents could not be used in conjunction with the standards recommended here.

Level-of-service standards are the very cornerstone of capacity analysis. It is believed that the recommendations made here result in a useful set of standards that both fulfill the requirements for such standards and more accurately reflect observed field conditions.

ACKNOWLEDGMENT

The research reported here is being conducted under contract to the Federal Highway Administration (FHWA). The cooperation of Harry Skinner in the presentation of this paper is appreciated. We also wish to thank Marvin Gersten of Howard Needles Tammen and Bergendoff and Francis A. Hyland of the Long Island State Park and Recreation Commission for providing data and allowing pilot field studies to be conducted.

The views presented here are ours. They do not necessarily represent the official views of FHWA or established policies or standard practices endorsed by FHWA.

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Effect of Trucks, Buses, and Recreational Vehicles on Freeway Capacity and Service Volume

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As part of a project sponsored by the Federal Highway Administration to revise and update Chapters 7, 8, and 9 of the 1965 Highway Capacity Manual, truck equivalents for specific grades have been recalibrated. The recalibration is based primarily on the results of freeway simulations conducted at Midwest Research Institute and studies of truck weight-to-power ratios and operating characteristics conducted at Pennsylvania State University. Approximate equivalents have also been developed for recreational vehicles, which form a significant portion of the traffic stream in selected areas.

The effect of trucks and buses on freeway flow is treated in the 1965 Highway Capacity Manual (HCM) (1) through the application of multiplicative correction factors to service volumes under ideal conditions drawn from Table 9.1 of the manual. The factors are based on automobile equivalents E_t or E_B, which represent the number of automobiles equivalent to one truck or bus under specified traffic and roadway conditions. Equivalents were calibrated by using a method developed for two-lane, two-way highways by Powell Walker. The manual (1), which uses the Walker method, states that

> for multilane highways, truck adjustment procedures are somewhat less well-defined, because the quantitative effect of trucks on the capacity of multilane highways for sustained grades is not as well known as it is for two-lane highways.

Multilane factors were eventually derived by manipulating the results of the California studies given in Chapter 5 of the HCM.

Since the publication of the 1965 HCM, a number of studies have been done on the effect of trucks on freeway flows, and others are in progress:

1. Simulation studies conducted by Midwest Research Institute (MRI) on the effect of trucks on freeway flow (2);
2. A study of the weight-to-power ratios of modern trucks and their operating characteristics conducted at Pennsylvania State University (3);
3. A study similar to the Pennsylvania State University work conducted in 1965 by Wright and Tignor (4);
4. The work of Werner and others on recreational vehicle and truck effects, primarily on two-lane highways (5); and
5. Unpublished studies of truck crawl speeds conducted by Rooney and Ching of the California Department of Transportation (DOT).

Under the sponsorship of the Federal Highway Administration (FHWA), we undertook to develop revised truck equivalents as well as similar equivalents for recreational vehicles. This paper presents the results of this work, which is based primarily on the results of the MRI and Pennsylvania State University studies mentioned above.

TRUCKS

MRI Simulations

A detailed simulation model of multilane highway flow that was developed and applied in a previous MRI contract was improved in a series of adjustments so that it duplicates the characteristics of mixed flows in level terrain and on grades. This model was adjusted and then validated by comparison with data collected on selected highway sites in California. Simulation results duplicate the important influences of grade, vehicle population, and flow rate for available cases.

The data collected for adjustment of the simulation model were taken at high flow rates on a 4 to 6 percent grade. In addition, data were collected on 2 percent grades. The parameters used in the simulation model included flow rate, distribution to lane by vehicle type, spot speeds, lane-changing frequencies, vehicle population, and overall travel speeds.

The simulation produces operating speed versus percentage capacity (V/C ratio) relations that would be observed in real traffic. Design charts were constructed by combining and interpreting the results from numerous simulation runs. The operating speed-percentage capacity relations were used to obtain an "implied capacity" for each simulation point (implied capacity is used because an actual test to obtain capacities has not been made at each location):

$$\text{Implied capacity} = \text{simulation flow rate} \div (\text{percentage capacity}/100) \qquad (1)$$

The combination of simulation runs is used to define implied capacity as a function of grade and percentage of commercial vehicles.

The resulting values of implied capacity are considerably higher than any capacities observed to date—some as high as 2600-2800 vehicles/h/lane. The simulations, however, were based on 3-min flows so that the implied capacities represent maximum 3-min flow rates, not full-hour volumes. Nevertheless, the variance of these numbers from generally accepted figures is a cause of some concern.

Design information includes the following parameters: number of lanes, design speed, grade, total flow rate, percentage of trucks, implied capacity, service level, operating speed, and percentage of implied capacity. All of these factors can be examined by using a family of design charts. Figure 1 is an example of a typical design chart for a four-lane freeway with a 112-km/h (70-mph) design speed based on typical automobile and truck populations.

The Typical Truck

There is some question as to what the deceleration and acceleration characteristics of the typical truck are on

Figure 1. Implied capacities versus percentage of trucks and sustained grade (two lanes, 112-km/h design speed).

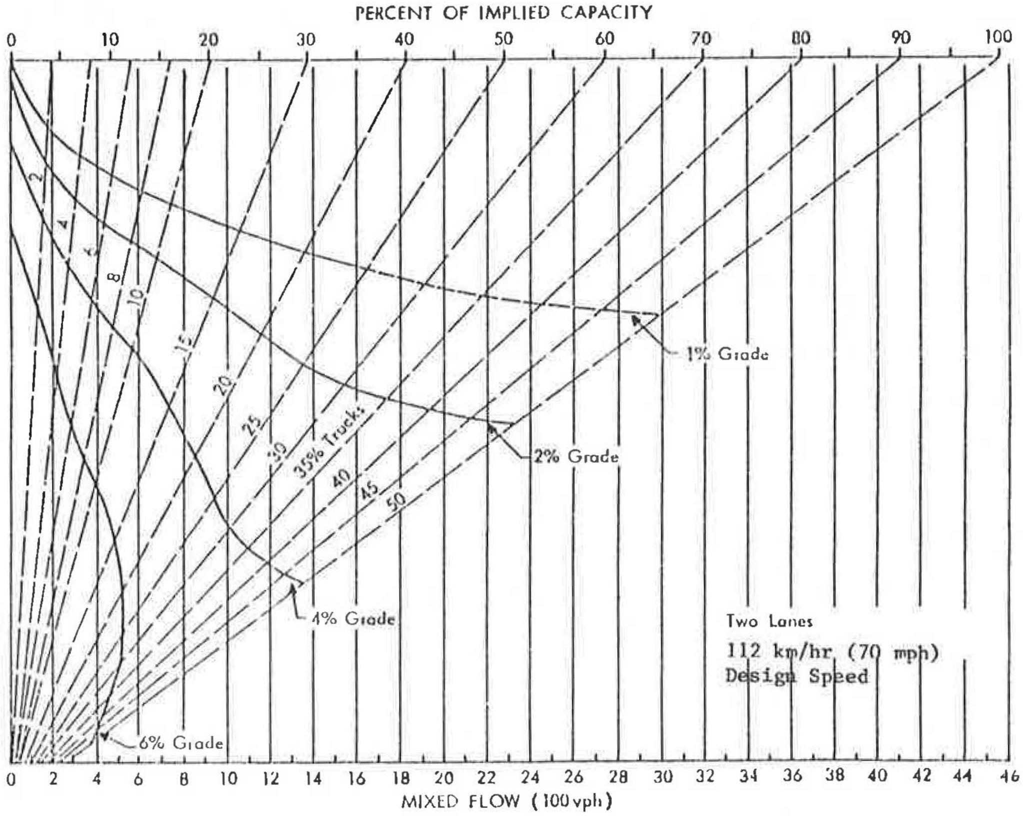


Table 1. Automobile equivalents for trucks on upgrades.

		E_t														
		Percentage Trucks on Four-Lane Freeways							Percentage Trucks on Six- or Eight-Lane Freeways							
Grade (%)	Length (m)	2	4	6	8	10	15	20	2	4	6	8	10	15	20
0	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	0-400	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	400-800	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	800-1200	4	4	4	3	3	3	3	4	4	3	3	3	3	3
	1200-1600	5	4	4	3	3	3	3	5	4	4	3	3	3	3
	1600-2400	6	5	5	4	4	4	3	6	5	4	4	4	3	3
	>2400	7	5	5	4	4	4	3	7	5	5	4	4	3	3
2	0-400	4	4	3	3	3	3	3	4	4	3	3	3	3	3
	400-800	7	5	5	4	4	4	4	7	5	5	4	4	4	4
	800-1200	8	6	5	5	4	4	4	8	6	5	5	4	4	4
	1200-1600	8	6	6	5	5	5	5	8	6	6	5	5	5	5
	1600-2400	9	7	7	6	6	5	5	9	7	6	5	5	5	5
	>2400	10	7	7	6	6	5	5	10	7	6	5	5	5	5
3	0-400	6	5	5	4	4	4	3	6	5	5	4	4	4	3
	400-800	9	7	6	5	5	5	5	8	7	6	5	5	5	5
	800-1200	12	8	7	6	6	6	6	10	8	6	5	5	5	5
	1200-1600	13	9	8	7	7	7	7	11	8	7	6	6	6	6
	>1600	14	10	9	8	8	7	7	12	9	8	7	7	7	7
4	0-400	7	5	5	4	4	4	4	7	6	5	4	4	3	3
	400-800	12	8	7	6	6	6	6	10	8	6	5	5	5	5
	800-1200	13	9	8	7	7	7	7	11	9	8	7	6	6	6
	1200-1600	15	10	9	8	8	8	8	12	10	9	8	7	7	7
	>1600	17	12	11	9	9	9	9	13	10	9	8	8	8	8
5	0-400	8	6	6	5	5	5	5	8	7	6	5	5	5	5
	400-800	13	9	8	7	7	7	7	11	8	7	6	6	6	6
	800-1600	20	15	14	11	11	11	11	14	11	10	9	9	9	9
	>1200	22	17	16	13	13	13	13	17	14	13	12	11	11	11
6	0-400	9	7	7	6	6	6	6	10	7	6	5	5	5	5
	400-800	17	12	11	9	9	9	9	13	10	9	8	8	8	8
	>800	28	22	21	18	18	18	18	20	17	16	15	14	14	14

Notes: 1 m = 3,3 ft.
Longest length category indicates equivalency at crawl speed.

modern multilane freeways, particularly with respect to those characteristics assumed in the MRI work.

Several different parameters determine the performance characteristics of motor vehicles. The most significant of these is the weight-to-power ratio. To determine the weight-to-power ratio of the typical truck, a search of the existing literature was undertaken. The following results were obtained:

1. A study conducted at Pennsylvania State University for NCHRP (3) used a 183-kg/kW (300-lb/hp) vehicle as their typical truck. This figure is based on information received from truck manufacturers and the operator of a major truck fleet.
2. The MRI study used for the generation of truck equivalents (2) uses a truck population with an average weight-to-power ratio of 138 kg/kW (225 lb/hp). A. D. St. John, one of the principal MRI researchers on this study, has indicated that the data collected in the study may not represent the typical situation on the nation's freeways and that the average truck probably has a higher weight-to-power ratio.
3. An MRI study of grade effects on traffic-flow stability and capacity (6) has indicated a population of trucks on grades with a typical vehicle of 183 kg/kW (48 percent of truck traffic on primary routes).

On the basis of this information, the 183-kg/kW vehicle was selected as the typical truck on which to base the generation of truck equivalents. Note that, as indicated in the Pennsylvania State University work, the crawl speeds of 183-kg/kW vehicles on grades are similar to those of 122-kg/kW (200-lb/hp) vehicles on multilane highways given in the 1965 HCM (1).

Since the MRI design charts were designed by using a concept called "percentage reference trucks", they can be used to generate truck factors for the 183-kg/kW (300-lb/hp) vehicle even though they were calibrated for MRI's typical truck population.

The concept of percentage reference trucks allows for the adjustment of any truck population to a common or reference base that can be used with the design charts. The relationship for this concept is

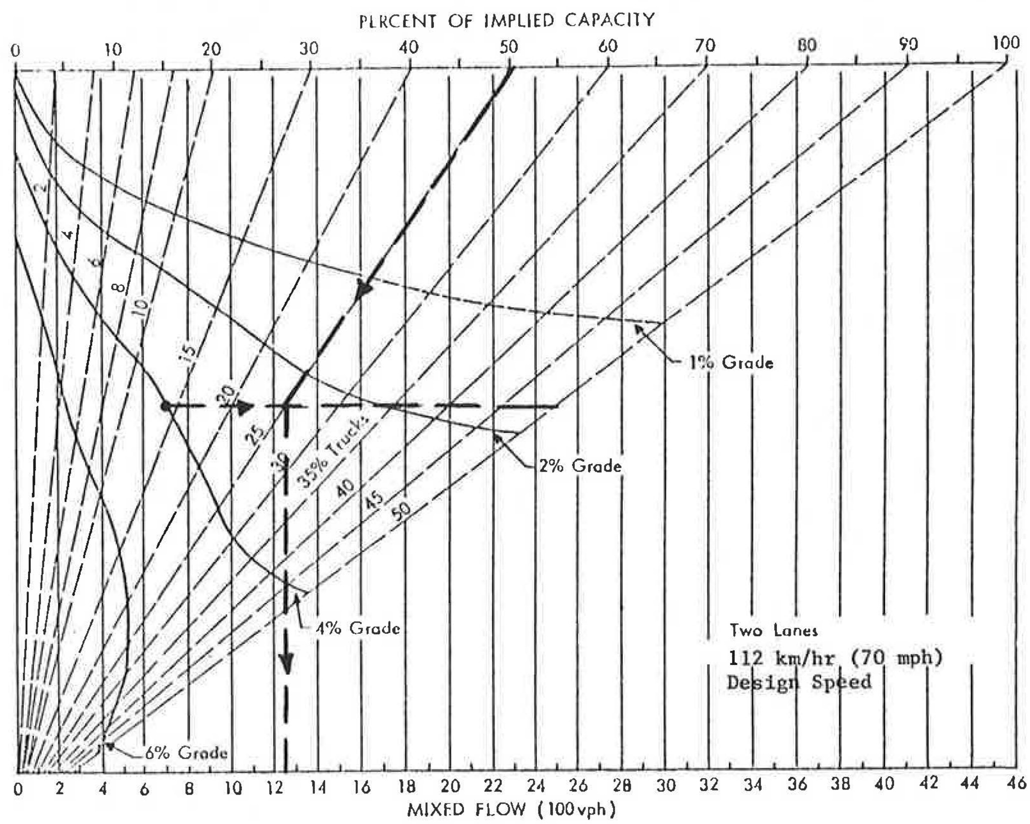
$$\text{Percentage reference trucks} = (100/F)(3.16f_{10} + 1.41f_9 + 0.14f_8 + 0.06f_7) \qquad (2)$$

where percentage reference trucks = percentage in terms of the reference population defined in Table 1, F = total flow rate of mixed vehicles, and f_i = flow rate of index number of trucks.

The 183-kg/kW (300-lb/hp) vehicle falls into the category of index 9 trucks. To use the MRI charts, it was assumed that F = 100 vehicles/h, $f_7 = f_8 = f_{10} = 0$, and f_9 = the percentage of trucks in the traffic stream. The table below gives the results of converting to percentage reference trucks (1 kg/kW = 1.63 lb/hp):

Typical 183-kg/kW Trucks (%)	Reference Trucks (%)
2	2.8
4	5.6
6	8.4
8	11.2
10	14.0
15	21.0
20	28.0

Figure 2. Computation for E_t based on MRI simulation.



It is the percentage reference trucks that is used to obtain automobile equivalents of trucks by using the MRI simulations.

Equivalents for Trucks on Sustained Grades

The following procedure is used to calculate automobile equivalents for any percentage of trucks on any severity of sustained grade (length of grade greater than or equal to the length at which the truck reaches its crawl speed for an indicated severity of grade) by using MRI simulation. The procedure is illustrated here for a specific case but has been applied in the generation of a complete table of equivalents.

Problem 1

Find automobile equivalents E_t for a traffic flow that consists of 10 percent trucks on a 4 percent sustained grade of a four-lane freeway where $V/C = 0.5$ and design speed = 112 km/h (70 mph).

Solution

1. Enter Figure 2 with 14 percent reference trucks and 4 percent grades. Find the point of intersection as shown in Figure 2. From this point, draw a horizontal line across the figure.
2. Enter Figure 2 on the "Percent of Implied Capacity axis with 0.5 and construct a line parallel to the fan of "% trucks" lines to the intersection of the line drawn in step 1.
3. Drop a vertical line from the point of intersection in step 2 to the mixed flow scale and read 1230 vehicles/h.
4. If

SV = service volume in automobiles/h implied by the chart in Figure 2 for $V/C = 0.5$,
Q = mixed flow (vehicles/h) (step 3),
Y = percentage trucks/100, and
E_t = automobile equivalents for one truck under the conditions specified for this problem,

then

$$SV = Q(1.00 - Y) + E_t YQ \quad (3)$$

and

$$E_t = [SV - Q(1.00Y)/YQ] \quad (4)$$

The value of SV, which is taken from Figure 2 so as to remain consistent with the simulation used to generate the charts, is found by taking the mixed-flow value that corresponds to 0 percent grade, 0 percent trucks, and 100 percent implied capacity and multiplying by V/C. In this case, $SV = 4550 \times 0.5 = 2275$ automobiles/h and $E_t = [2275 - 1230\ (0.90)]/\ [(0.10)\ (1230)] = 9.5$.

This procedure has been used to generate automobile equivalents for a wide selection of combinations of truck traffic and sustained grade. It can generate a set of truck factors for various design speeds, V/C ratios, and numbers of lanes. From these sample calculations, it was found that changing the V/C ratio or freeway design speed does not change the equivalent significantly. The size (number of lanes) of the freeway, however, does prove to be important in cases of a high percentage of trucks and/or a steep grade. It appears that there is justification for calibrating equivalents separately for two lanes and for freeways with three or more lanes. As the number of lanes increases, the difference in the effect of trucks on flow should stabilize. Since the MRI method does not treat freeways that have more than six lanes, truck factors are computed for four-lane freeways and for freeways with six or more lanes.

It is critical to note the meaning of truck equivalents that are computed in this way. The resulting truck equivalents will convert a service volume in automobiles per hour (from Table 9.1 of the 1965 HCM or equivalent) to a volume in mixed vehicles per hour that will consume the same percentage of roadway capacity. Thus, truck equivalents are based on keeping constant the effective value of V/C for any given level of service.

The procedure described above led to the calculation of automobile equivalents of trucks on sustained grades. Equivalents for lengths of grades on which the crawl speeds of trucks have not yet been reached must be computed differently.

Equivalents for Trucks on Grades Shorter Than Critical Length

Deceleration curves for a 183-kg/kW (300-lb/hp) vehicle are shown in Figure 3 (3). The Pennsylvania State University curves compare favorably with those presented in the MRI report for the index 9 truck [153-215 kg/kW (250-350 lb/hp) vehicle]. The difference in these curves is the speed of trucks on a level grade. The Pennsylvania curves assume a speed of 88 km/h (55 mph) on a level grade compared with 70 km/h (44 mph) in the MRI study. But St. John of MRI indicates that the 70-km/h speeds are lower than average.

The 88-km/h speed and the Pennsylvania curves are used here because informal observations have indicated that trucks keep up with the flow of automobile traffic in situations of 0 percent grade on freeways. In fact,

Figure 3. Speed-distance curves for a typical heavy truck of 183 kg/kW.

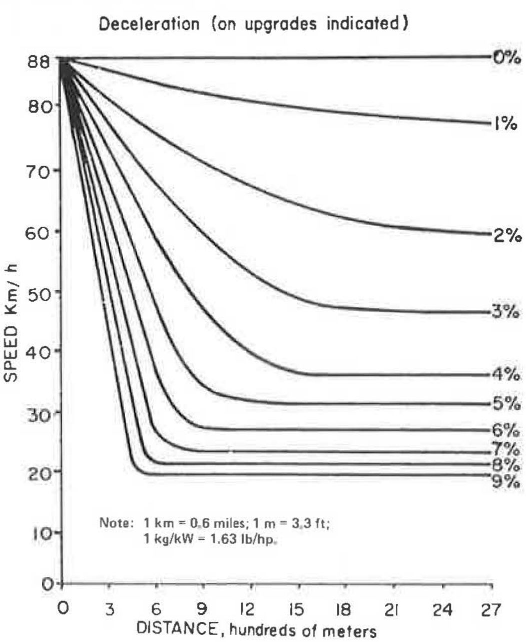


Table 2. Values of E_t for heavy-truck populations.

Grade (%)	Length (m)	E_t Percentage Trucks on Four-Lane Freeways 2	4	6	8	10	15	20	Percentage Trucks on Six- or Eight-Lane Freeways 2	4	6	8	10	15	20
0	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	0-400	4	3	3	3	3	3	3	4	3	3	3	3	3	3
	400-800	5	4	4	4	4	3	3	5	4	4	4	4	3	3
	800-1200	7	5	5	4	4	4	4	7	5	4	4	4	4	4
	1200-1600	8	6	6	5	5	4	4	8	6	5	5	5	4	4
	1600-2400	10	7	6	5	5	4	4	10	7	6	5	5	4	4
	>2400	11	8	7	6	6	5	5	11	8	7	6	6	5	5
2	0-400	8	6	6	5	5	4	4	7	5	5	5	5	4	4
	400-800	10	7	7	6	6	5	5	9	6	6	6	6	5	5
	800-1200	12	9	8	8	7	6	6	11	8	7	7	7	6	6
	1200-1600	14	10	9	9	8	7	7	13	9	8	8	7	6	6
	1600-2400	16	11	9	9	8	8	8	15	10	9	9	8	7	7
	>2400	16	12	10	10	9	8	8	15	11	10	9	8	7	7
3	0-400	11	10	9	8	8	7	7	9	8	8	7	7	6	6
	400-800	13	12	11	9	9	8	8	11	10	9	8	8	7	7
	800-1200	16	14	12	11	10	10	10	13	12	11	10	9	8	8
	1200-1600	19	15	14	13	12	12	12	16	13	13	12	11	10	10
	>1600	22	16	15	15	14	14	14	18	14	14	13	12	11	11
4	0-400	13	11	10	10	9	8	8	11	9	9	9	8	8	8
	400-800	18	13	13	12	12	12	12	13	11	11	11	10	9	9
	800-1200	22	15	15	14	14	14	14	16	13	13	13	12	11	11
	1200-1600	24	18	18	17	17	17	17	19	15	15	15	14	13	13
	>1600	26	20	19	19	19	19	19	21	17	17	16	16	14	14
5	0-400	19	16	16	16	16	16	16	17	13	12	12	12	11	11
	400-800	26	21	21	21	21	21	21	22	17	16	16	16	15	15
	800-1200	33	27	27	27	27	27	27	27	21	20	20	20	19	19
	>1200	40	32	32	32	32	32	32	31	25	24	24	24	23	23

Note: 1 m = 3.3 ft.

Table 3. Values of E_t for light-truck populations.

Grade (%)	Length (m)	E_t Percentage Trucks on Four-Lane Freeways 2	4	6	8	10	15	20	Percentage Trucks on Six- or Eight-Lane Freeways 2	4	6	8	10	15	20
0	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2
2	0-1200	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	>1200	3	3	3	3	3	3	3	3	2	2	2	2	2	2
3	0-400	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	400-800	3	3	3	3	3	3	3	3	2	2	2	2	2	2
	800-1200	4	4	3	3	3	3	3	3	3	3	3	3	2	2
	1200-1600	4	4	3	3	3	3	3	3	3	3	3	3	3	3
	1600-2400	5	5	4	4	4	3	3	4	3	3	3	3	3	3
	>2400	6	5	4	4	4	3	3	4	3	3	3	3	3	3
4	0-400	3	2	2	2	2	2	2	3	2	2	2	2	2	2
	400-800	5	3	3	3	3	3	3	3	3	3	3	3	3	3
	800-1200	6	4	4	3	3	3	3	4	3	3	3	3	3	3
	1200-2400	7	5	5	4	4	3	3	4	4	3	3	3	3	3
	>2400	8	6	5	4	4	3	3	5	4	3	3	3	3	3
5	0-400	4	3	3	3	3	3	3	4	3	3	3	3	3	3
	400-800	6	4	4	4	4	3	3	5	4	3	3	3	3	3
	800-1600	7	5	5	4	4	3	3	6	4	3	3	3	3	3
	1600-2400	9	6	6	4	4	3	3	7	5	4	4	4	3	3
	>2400	12	8	7	5	5	4	4	8	6	4	4	4	3	3
6	0-400	5	4	3	3	3	3	3	4	4	3	3	3	3	3
	400-800	8	6	5	4	4	3	3	6	5	4	3	3	3	3
	800-1600	12	8	7	5	4	3	3	8	6	4	4	4	3	3
	>1600	16	10	8	6	5	4	4	10	7	5	4	4	3	3

Note: 1 m = 3.3 ft.

this observation has led to the HCM automobile equivalent of 2 for trucks on 0 percent grade, which is based on the greater space that trucks need and the larger headways they command.

Truck equivalency is based on the premise that trucks travel slower than automobiles on grades. Their deceleration curves can therefore be used to obtain automobile equivalents for vehicles that have not yet reached their crawl sped if one equates the speed of the truck to that of a similar vehicle on a grade on which the indicated speed would be the crawl speed. The E_t for this grade is then used as the truck equivalent. The problem and solution given below illustrate this procedure.

Problem 2

Assuming that the 10 percent truck population of problem 1 has proceeded 600 m (2000 ft) along the grade, find the automobile equivalent of any truck.

Solution

1. Enter Figure 3 with length of grade = 600 m and intersect curve for 4 percent grade. Read speed = 57.6 km/h (36 mph). This is almost the same speed as the crawl speed of trucks on a 2 percent grade.

2. Enter Table 1 with 10 percent trucks and 2 percent grade (at crawl speed) and find E_t = 6.

By using this procedure and the design charts of the MRI report, a complete set of automobile equivalents is generated. These are given in Table 1.

To account for instances in which the truck population may not be typical, truck equivalents were also computed for light trucks [those with an average weight-to-power ratio of 92 kg/kW (150 lb/hp)] and for heavy trucks [those with ratios higher than 215 kg/kW (350 lb/hp)]. These are given in Tables 2 and 3.

Some of the values in Table 1 are unusual in that they tend to indicate that equivalents decrease as the percentage of trucks increases beyond 10 percent. This is not totally unreasonable. In fact, values of E_t in Table 9.4 of the 1965 HCM show a similar trend. At high truck percentages, trucks tend to separate from other traffic. Thus, their flow becomes less disruptive. Although the cumulative effect continues to increase, the effect of each truck decreases.

RECREATIONAL VEHICLES

Since recreational vehicles are taking on added importance on the nation's highways, it would be desirable to develop a set of automobile equivalents (E_R) for these vehicles. Although Werner has done work in this area (7), the discussion here is based primarily on Walker's methodology and is exclusively for two-lane highways.

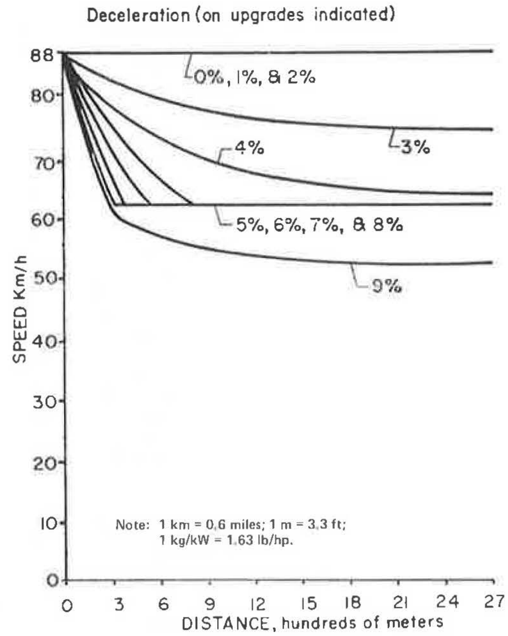
In the approach used here to generate representative automobile equivalents for recreational vehicles, the Pennsylvania State University deceleration curves for a 37-kg/kW (60-lb/hp) vehicle [Figure 4 (3)] and the truck equivalents previously computed are used. Values of E_R have been developed based on the speed of the recreational vehicle at various points along a grade. These speeds are found from the Pennsylvania curves for a weight-to-power ratio of 37 (60). The position of a truck with an equivalent speed is found on the Pennsylvania truck curves, and the appropriate E_R is selected. This technique is approximate and does not account for the differing driver characteristics for trucks and recreational vehicles, but it is the best that can be formulated given the extant data base. The values computed for E_R are given in Table 4.

It is recommended that such equivalents be used. Recreational vehicles take on great importance in certain areas of the country. Using even approximate E_R values would be better than not accounting for such vehicles at all or assuming that they are trucks. Further, the existence of E_R values in a formal document such as the HCM may spur additional research efforts in this area.

BUSES

Literature on the subject of bus equivalents and bus operating characteristics is virtually nonexistent. Thus, it appears that the values of E_B given in Tables 9.3a (generalized sections) and 9.5 (specific grades) in the 1965 HCM (1) should be continued.

Figure 4. Speed-distance curves for a typical trailer combination of 37 kg/kW.



FACTORS

Tables of the form given in the 1965 HCM can be used for the conversion of E_t, E_R, and E_B to factors that reflect the impact of these vehicles on traffic flow. Table 5 (1) in this paper can be used in the case of a population of automobiles and trucks only (or automobiles and recreational vehicles only or automobiles and buses only). In the case of a population in which automobiles, trucks, recreational vehicles, and buses are all present in significant percentages, a commercial vehicle factor should be computed from the following formula:

$$C = 100/(100 - P_t - P_R - P_B + P_tE_t + P_RE_R + P_BE_B) \quad (5)$$

where

C = adjustment factor;

P_t, P_R, P_B = percentage of trucks, recreational vehicles, and buses, respectively, in the traffic stream; and

E_t, E_R, E_B = automobile equivalents of trucks, recreational vehicles, and buses, respectively, in the traffic stream.

By using this combined factor, service volumes may be corrected for the combined effect of vehicles other than automobiles in the traffic stream:

$$SV = MSV \times \underline{C} \times W \quad (6)$$

It is recommended that this combined commercial vehicle factor be used in all cases where buses and recreational vehicles are present in quantities significant enough to be separately considered. Where only trucks are considered, a table that converts E_t to a factor (Table 5) may be used. Development of a nomograph to simplify the computation of C is being investigated.

Table 4. Values of E_R on upgrades.

Grade (%)	Length (m)	E_R: Percentage Recreational Vehicles on Four-Lane Freeways: 2	4	6	8	10	15	20	Percentage Recreational Vehicles on Six- or Eight-Lane Freeways: 2	4	6	8	10	15	20
0-2	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2
3	0-400	3	2	2	2	2	2	2	3	2	2	2	2	2	2
	400-800	4	3	3	3	3	3	3	4	3	3	3	3	3	3
	800-1200	6	4	3	3	3	3	3	5	4	3	3	3	3	3
	1200-1600	7	5	4	4	4	4	4	6	5	4	4	4	4	4
	>1600	7	5	5	4	4	4	4	6	5	5	4	4	4	4
4	0-400	5	4	4	4	4	3	3	5	4	4	3	3	3	3
	400-800	7	5	5	4	4	4	4	6	5	4	3	3	3	3
	800-1200	8	6	5	4	4	4	4	6	5	4	3	3	3	3
	1200-1600	9	7	6	5	5	4	4	7	6	5	4	4	4	4
	>1600	9	7	6	5	5	4	4	7	6	5	4	4	4	4
5	0-400	5	4	4	4	4	3	3	5	4	4	4	4	3	3
	400-800	8	6	6	5	5	4	4	7	6	5	4	4	4	4
	>800	10	7	7	6	6	5	5	10	7	6	5	5	5	5
6	0-400	5	4	4	4	4	3	3	5	5	5	4	4	3	3
	400-800	10	7	7	6	6	5	5	10	7	6	5	5	5	5
	>800	10	7	7	6	6	5	5	10	7	6	5	5	5	5

Note: 1 m = 3.3 ft.

Table 5. Adjustment factors where only one type of nonautomobile vehicle is present in significant percentages.

Automobile Equivalent[a]	Adjustment Factor $C^{b,c}$ by Percentage of Trucks, Buses, or Recreational Vehicles: 1	2	3	4	5	6	7	8	9	10	12	14	16	18	20
2	0.99	0.98	0.97	0.96	0.95	0.94	0.93	0.93	0.92	0.91	0.89	0.88	0.86	0.85	0.83
3	0.98	0.96	0.94	0.93	0.91	0.89	0.88	0.86	0.85	0.83	0.81	0.78	0.76	0.74	0.71
4	0.97	0.94	0.92	0.89	0.87	0.85	0.83	0.81	0.79	0.77	0.74	0.70	0.68	0.65	0.63
5	0.96	0.93	0.89	0.86	0.83	0.81	0.78	0.76	0.74	0.71	0.68	0.64	0.61	0.58	0.56
6	0.95	0.91	0.87	0.83	0.80	0.77	0.74	0.71	0.69	0.67	0.63	0.59	0.56	0.53	0.50
7	0.94	0.89	0.85	0.81	0.77	0.74	0.70	0.68	0.65	0.63	0.58	0.54	0.51	0.48	0.45
8	0.93	0.88	0.83	0.78	0.74	0.70	0.67	0.64	0.61	0.59	0.54	0.51	0.47	0.44	0.42
9	0.93	0.86	0.81	0.76	0.71	0.68	0.64	0.61	0.58	0.56	0.51	0.47	0.44	0.41	0.38
10	0.92	0.85	0.79	0.74	0.69	0.65	0.61	0.58	0.55	0.53	0.48	0.44	0.41	0.38	0.36
11	0.91	0.83	0.77	0.71	0.67	0.63	0.59	0.56	0.53	0.50	0.45	0.42	0.38	0.36	0.33
12	0.90	0.82	0.75	0.69	0.65	0.60	0.57	0.53	0.50	0.48	0.43	0.39	0.36	0.34	0.31
13	0.89	0.81	0.74	0.68	0.63	0.58	0.54	0.51	0.48	0.45	0.41	0.37	0.34	0.32	0.29
14	0.88	0.79	0.72	0.66	0.61	0.56	0.52	0.49	0.46	0.43	0.39	0.35	0.32	0.30	0.28
15	0.88	0.78	0.70	0.64	0.59	0.54	0.51	0.47	0.44	0.42	0.37	0.34	0.31	0.28	0.26
16	0.87	0.77	0.69	0.63	0.57	0.53	0.49	0.45	0.43	0.40	0.36	0.32	0.29	0.27	0.25
17	0.86	0.76	0.68	0.61	0.56	0.51	0.47	0.44	0.41	0.38	0.34	0.31	0.28	0.26	0.24
18	0.85	0.75	0.66	0.60	0.54	0.49	0.46	0.42	0.40	0.37	0.33	0.30	0.27	0.25	0.23
19	0.85	0.74	0.65	0.58	0.53	0.48	0.44	0.41	0.38	0.36	0.32	0.28	0.26	0.24	0.22
20	0.84	0.72	0.64	0.57	0.51	0.47	0.42	0.40	0.37	0.34	0.30	0.27	0.25	0.23	0.21
21	0.83	0.71	0.63	0.56	0.50	0.45	0.41	0.38	0.36	0.33	0.29	0.26	0.24	0.22	0.20
22	0.83	0.70	0.61	0.54	0.49	0.44	0.40	0.37	0.35	0.32	0.28	0.25	0.23	0.21	0.19
23	0.82	0.69	0.60	0.53	0.48	0.43	0.39	0.36	0.34	0.31	0.27	0.25	0.22	0.20	0.19
24	0.81	0.68	0.59	0.52	0.47	0.42	0.38	0.35	0.33	0.30	0.27	0.24	0.21	0.19	0.18
25	0.80	0.67	0.58	0.51	0.46	0.41	0.37	0.34	0.32	0.29	0.26	0.23	0.20	0.18	0.17

[a] Computed by $100/(100 - P_R + E_R P_R)$ or $100/(100 - P_a + E_a P_a)$ (1, Ch. 5). Use this formula for larger percentages.
[b] From HCM, Table 9.4 or Table 9.5 (1).
[c] Trucks and buses should not be combined in entering this table where separate consideration of buses has been established as required because automobile equivalents differ.

COMPOSITE GRADES

In the 1965 HCM, composite grades are normally accounted for by finding truck and bus equivalents based on the average grade. Thus, equivalents for a 2 percent upgrade of 300 m (1000 ft) followed by a 4 percent upgrade of 300 m are computed as if for a 3 percent grade of 600 m (2000 ft).

Leisch (8) has developed a more exact technique that uses typical acceleration and deceleration curves for a truck to determine the actual speed of a truck at any point along a composite grade. For lengthy composite grades, the difference between the HCM technique and that of Leisch can be significant. It is recommended, therefore, that the Leisch method be included in the freeway procedures being developed as an alternative where composite grades of many sections or great length are involved. Guidelines for when to use it and when to rely on the simpler average grade approach should also be developed. The Pennsylvania State University deceleration and acceleration curves can be used to analyze these composite sections for truck and recreational vehicle traffic.

Research into the effect of nonpassenger vehicles on freeway downgrades is sparse. The MRI work contains some downgrade simulations, but these are not detailed enought to permit the generation of downgrade factors. The HCM recommends that freeway downgrades be treated as level grades in the absence of specific performance data on downgrade operations. This is reasonable except where trucks and other vehicles are forced to shift into lower gears. Procedures now being developed would caution users on this point and would present general recommendations for handling it.

CONCLUSIONS AND RECOMMENDATIONS

The automobile equivalents discussed in this report were obtained from the best available information on this subject. More research on the topic is needed, however, especially in the areas of recreational vehicles and buses and downgrade effects. It would also be of interest to see studies conducted on the effect on traffic flow of truck populations composed of vehicles with different performance characteristics. The MRI concept of percentage reference trucks presents a good base for future studies of this kind.

ACKNOWLEDGMENT

The research reported here is being conducted under the sponsorship of FHWA. The cooperation of Harry Skinner in the presentation of this paper is appreciated.

The views presented here are ours. They do not necessarily represent the official views of FHWA or established policies or standard practices endorsed by FHWA.

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Discussion

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The speeds presented in the paper by Linzer, Roess, and McShane for typical trucks on grades are much slower than the speeds of typical trucks on grades along rural freeways and expressways in California.

The speeds of more than 14 000 trucks and 2600 recreational vehicles, pickup trucks, vans, and other vehicles were measured on grades along rural freeways and expressways in California during 1977, 1978, and 1979. Speed measurements were obtained during free-flow traffic conditions when wind velocities were 3.6 m/s (7 knots) or less. Speeds were not recorded for trucks that were following other trucks along a lane at intervals of less than 7 s. The speeds of trucks were measured without regard to whether the trucks were empty, partially loaded, or loaded.

The horizontal alignment at all locations where speeds were measured is suitable for high speeds. All upgrades where sustained speeds were measured, except the 4.0 percent grade, are over 3.2 km (2 miles) in length.

The measured speeds along the 4.0 percent grade were not sustained speeds. The distance from the beginning of the grade near a truck scale to the location where the speeds were measured is only 2.1 km (1.3 miles). Loaded trucks were required to slow to 5 km/h (3 mph), and empty trucks were required to slow to 8 km/h (5 mph) at this truck scale. This apparently affected the measured average speed of five-axle trucks by approximately 1 km/h (0.6 mph) and the 12.5 percentile speed of five-axle trucks by approximately 2 km/h (1.2 mph). The deceleration measurements were obtained along a 4.0 percent grade at a different location where there is not a truck scale.

Measured speeds along the 6.0 percent grade were slightly affected by variable grades in advance of the location where speed measurements were obtained. These variable grades did not cause the measured speeds to differ much from sustained speeds. The measured speeds along this grade are therefore referred to in this discussion as sustained speeds.

The table below gives average sustained speeds along grades for all trucks (both trucks and truck combinations are referred to as trucks in this discussion). The speeds were calculated, by using the measured speeds, for 15 percent two-axle trucks, 5 percent three-axle trucks, 5 percent four-axle trucks, and 75 percent five-axle trucks. These are typical percentages along the rural freeways and expressways where the speed measurements were obtained. The measured speeds of 6400 trucks were used in preparing the table (1 km/h = 0.6214 mph):

Grade (%)	Speed (km/h)	Grade (%)	Speed (km/h)
1.78	82.74	5.0	58.02
3.0	71.89	6.0	52.29
4.0	63.23	7.0	49.33

The following table gives average sustained speeds along grades for five-axle trucks (measured speeds of 4900 trucks were used):

Grade (%)	Speed (km/h)	Grade (%)	Speed (km/h)
1.78	80.98	5.0	56.15
3.0	69.73	6.0	51.64
4.0	61.14	7.0	48.55

The next table gives 12.5 percentile sustained speeds along grades for all trucks:

Grade (%)	Speed (km/h)	Grade (%)	Speed (km/h)
1.78	70.20	5.0	38.70
3.0	53.91	6.0	30.79
4.0	42.39	7.0	26.30

Speeds given in these three tables for the 4.0 percent grade are not sustained speeds.

The following table gives average speeds of five-axle trucks decelerating along upgrades [the measurements were obtained at 152-m (500-ft) intervals, and the speeds of a minimum of 150 trucks were measured at each location]:

Speed (km/h)

2.88 Percent Grade	4.0 Percent Grade	5.0 Percent Grade	5.89-6.0 Percent Grade
87.60	88.31	88.63	69.40 (5.89 %)
85.38	83.80	83.19	63.04 (5.89 %)
83.03	79.21	77.04	58.00
81.43	75.96	73.05	55.81
79.94	72.79	68.70	54.83
77.49	69.85	65.32	54.04
76.85	66.24	61.59	53.30
75.95	63.17	59.53	
75.06	61.96	57.53	

Initial speed measurements were made near the beginning of each grade, and final measurements were made where average speeds were near the average sustained speeds previously determined.

The table below gives average speeds of five-axle trucks along 1372 m (4500 ft) of a -0.14 percent grade near a truck scale. Loaded trucks were required to slow to 5 km/h (3 mph), and empty trucks were required to slow to 8 km/h (5 mph) at this scale. The speeds of 100 trucks were measured at each of the first 10 locations, and the speeds of 150 trucks were measured at each of the last 3 locations (1 m = 3.3 ft; 1 km = 0.6214 mile):

Distance From Scale (m)	Speed (km/h)	Distance From Scale (m)	Speed (km/h)
30	16.06	610	67.48
61	22.92	762	72.32
91	27.84	914	76.99
122	34.05	1067	78.81
152	37.21	1219	81.59
305	51.45	1372	84.93
457	59.72		

Speed information was also obtained along a 3.0 percent grade and a 4.0 percent grade (farming area to urban area and return) when there were a significant number of agricultural trucks traveling. The measured average speeds of five-axle trucks were 1.67 km/h (1.04 mph) slower along the 3.0 percent grade and 2 km/h (1.24 mph) faster along the 4.0 percent grade when there were a significant number of agricultural trucks traveling. The difference in speeds between the grades was apparently caused by whether the agricultural trucks were loaded or empty.

The paper by Linzer, Roess, and McShane includes development of truck equivalency factors but does not include measurements of the actual speeds of trucks on grades. The truck equivalency factors were calculated by using information from a report prepared by Pennsylvania State University and information from the Midwest Research Institute. Again, the measured speeds of typical trucks on grades along rural freeways and expressways in California are much faster than the calculated speeds of the typical truck used in the paper.

Truck characteristics should be measured at various locations. The best procedure would be to measure the sustained speeds of trucks on various grades. Another procedure might be to measure truck accelerations near locations such as truck scales.

ACKNOWLEDGMENT

We wish to thank Howard K. Fong for checking the calculations.

The research project from which our data are taken was conducted as part of the Highway Planning and Research work program of the California DOT and FHWA.

The contents of this Discussion reflect our views, and we are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the state of California or FHWA. This discussion does not constitute a standard, specification, or regulation.

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In addition to the studies referenced in the paper by Linzer, Roess, and McShane, independent studies of the speed of trucks and recreational vehicles on grades were conducted in Texas in 1973 and 1974 (9, 10). Speed histories of 431 trucks and 260 recreational vehicles operating on grades between 2 and 7 percent were developed from direct field observations, and predictive equations relating several vehicle and driver characteristics were formulated through a stepwise regression analysis. A total of 11 factors were included in the analysis. Although weight-to-power ratio was found to have a significant effect on vehicle performance on grades, as noted by the authors, other factors such as entering speed, length and percentage of grade, and driver behavior also affected the speed history of both trucks and recreational vehicles.

It is interesting to note that the speed-distance relations selected by the authors for a typical heavy truck (Figure 3) agree within about 10 percent on upgrades up to 450 m (1500 ft) long with such composite curves for the typical heavy truck recommended by Walton and Lee for climbing lane design (10, Figure 8). These relations apply only to trucks entering the upgrade at 89 km/h (55 mph). Similarly, there is very good agreement between the respective curves shown in Figure 4 and those of Walton and Lee (9, Figure 48), which describe the speed-distance relation for typical recreational vehicles operating on 0 to 6 percent grades as long as about 600 m (2000 ft) after the vehicles enter the grade at 89 km/h. Again, these relations apply only for the specific entry speed. The Texas data therefore support the authors' selected vehicle performance data for these representive conditions.

The effects of vehicles entering upgrades at speeds other than 89 km/h are apparently not evaluated in the development of the new equivalency factors. The Texas observations indicate that entry speed has a considerable effect on deceleration rates for both trucks and recreational vehicles.

We commend Linzer, Roess, and McShane for their pragmatic approach to revising equivalency factors so that engineers can account for the changes that have occurred during the past two decades in vehicle performance and in the composition of the mixed traffic stream. The authors' assumptions concerning the performance of typical vehicles appear to be reasonable, and their use of previously accepted research results is innovative.

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Street Capacity for Buses in the Honolulu Central Business District

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A bus demonstration conducted in January 1978 in Honolulu is reported. The purpose of the demonstration was to determine the bus capacity of Hotel Street, the major bus corridor in the Honolulu central business district, under existing traffic and roadway conditions. Although buses were metered into both directions of Hotel Street at flow rates of 60, 120, 138, and 150 buses/h, only 100 to 120 buses/h could actually enter the system. Restrictions within the system further reduced bus flow. Major bottlenecks are identified, and the resulting impacts on vehicles, pedestrians, and the environment are assessed. It is concluded that directional bus capacity on Hotel Street was 95-100 buses/h at average speeds that ranged from 3 to 5 km/h (2 to 3 mph).

A major transit trip generator in Honolulu is the central business district (CBD), which encompasses an area of about 0.5 km^2 (0.2 mile^2). This generator is served by 22 of the 39 available scheduled bus routes. The primary east-west roadway used by bus routes through the CBD is Hotel Street, which is approximately 0.8 km (0.5 mile) long and is intersected by nine one-way side streets, seven of which are signalized (see Figure 1). There are 10 bus stops along Hotel Street, 6 on the north side and 4 on the south side. Fifteen of the 22 bus routes use some section of Hotel Street, and 7 bus routes intersect Hotel Street. During the off-peak period, Hotel Street handles between 50 and 56 buses/h in each direction. This increases to 72-80 buses/h during the morning peak period.

Hotel Street is a two-lane collector approximately 11 m (36 ft) wide that serves mixed traffic. At some intersections, the roadway flares to 12 m (40 ft), which allows both left and through movements in one lane. Although there are no bus bays on Hotel Street, it is not unusual for vehicles to pass one or two buses loading or unloading at a bus stop.

The land use adjacent to Hotel Street is zoned B-4, CBD, which is intended to denote the metropolitan center for financial, commercial, government, professional, and cultural activities. Also in the surrounding area are the state capitol, city hall, government offices, and major tourist attractions of historical interest.

In terms of transit, the city and county of Honolulu currently maintains a fleet of 350 buses. The system is wholly owned by the city and county of Honolulu, but its operation is contracted to a private carrier—MTL. This bus system is well received in the community. Although the urban portion of Honolulu ranks forty-third in population, bus ridership is the thirteenth highest in the country. Ridership figures for 1977 indicate a total ridership of 66.6 million, composed of 47.5 million paying passengers, 11.8 million transfers, and 7.3 million free senior citizen passengers.

PURPOSE OF THE STUDY

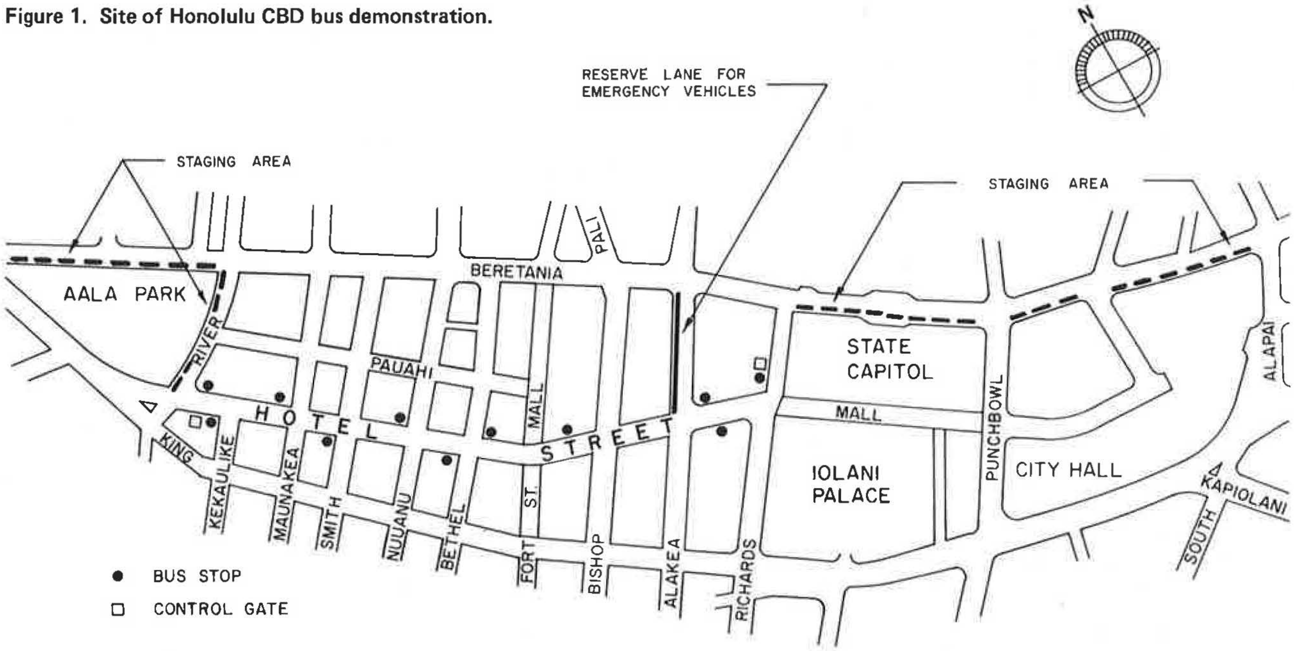
On January 20, 1978, the Honolulu Department of Transportation Services (DTS) conducted a study that involved the regulation of the major bus flow through the Honolulu CBD. The purpose of the Hotel Street bus demonstration was to determine the maximum bus volume Hotel Street can carry under present roadway and traffic conditions. The existing literature (1, 2, 3) indicates a wide range of values. The study also attempted to identify major bottlenecks and to quantify the resulting impacts on vehicles, pedestrians, and the environment.

The bus study was conducted under two constraints. First, the study occurred on Friday between 10:00 a.m. and 12:30 p.m., during the normal work periods of the department staff. Because of this, the observed traffic measures do not reflect peak-hour traffic conditions that occur on Hotel Street. Second, efforts were made to maintain current patterns of automobile use and bus patronage. Traffic signal timings and bus routes were not changed for the study.

PROCEDURE

During the bus demonstration, the flow of buses into Hotel Street was controlled in both directions. During various phases of the 10:00 a.m. to 12:30 p.m. test period, buses were scheduled to enter both ends of Hotel Street

Figure 1. Site of Honolulu CBD bus demonstration.



at predetermined flow rates.

Given a supply of 60 test buses and an off-peak directional volume of approximately 55 vehicles/h, bus flow rates greater than 120 buses/h in each direction could not be sustained for very long. This factor, plus the constraint of minimizing traffic impacts when possible, resulted in each of the four test periods lasting no more than 30 min.

Four bus flow rates, or headways, were used in the study. From 10:00 to 10:30 a.m., buses that entered Hotel Street had an initial headway of 60 s, which corresponds to an equivalent flow of 60 buses/h. This approximated the existing off-peak bus volume even though current bus traffic does not enter Hotel Street with uniform headways. Between 10:35 and 11:00 a.m., buses were scheduled to enter at a rate of 120 buses/h (30-s headway). Between 11:05 and 11:35 a.m., buses were scheduled to enter at a rate of 138 buses/h (26-s headway) and, between 12:00 noon and 12:30 p.m., buses were scheduled to enter every 24 s or at a flow rate of 150 buses/h.

To determine the bus capacity of Hotel Street, the controlling intersection with the minimum service volume was identified for each direction. The maximum bus volume passing this intersection during the 30-, 26-, and 24-s-headway test periods was defined as the capacity on Hotel Street for the respective direction.

To regulate flow rates entering both ends of the system, control gates were set up eastbound on Hotel at River and westbound on Richards at Hotel (Figure 1). The 60 additional test buses used to maintain the desired constant headways were stored in staging areas near the control gates and released into the system when they were needed.

Two scheduled bus routes entered and departed from Hotel Street at nonregulated points. These buses were monitored while they were on Hotel Street.

To maintain the actual pattern of bus patronage, boarding and alighting times of regularly scheduled buses were not controlled. However, test buses were required to stop for 15 s at each bus stop to simulate passenger boarding and alighting. Current policy allows the first two buses at a bus stop to load or unload their passengers and requires the following buses in the queue to wait until they reach the first two positions. This policy was maintained during the bus study.

While the experiment was being conducted, traffic, pedestrian, and environmental data were collected to assess transportation impacts. More than 150 persons from various city and state agencies were involved in the experiment. Buses were controlled and bus movements past each bus stop and intersection were monitored by 109 persons, including 60 test bus drivers. To assess impacts on automobile movements, 23 persons collected data for various traffic studies. Six persons recorded pedestrian movements at key sites crossing Hotel Street. To assess environmental impacts, four persons recorded noise levels and another four recorded air pollution levels. Six persons photographed the demonstration.

Before conducting the Hotel Street bus demonstration, DTS met with a number of local agencies to ensure the safety of the public during the study period, and an emergency plan was established.

FINDINGS

A brief discussion of the findings of the study is presented here. A more detailed discussion, with photographs, is available elsewhere (4).

Bus Movements

During the test periods, bus travel times through the system could be categorized into three phases. The first was the lag phase, which usually occurred during the beginning of the test period. This phase is characterized by relatively short travel times through the system. The buses at the beginning of the lag phase seemed to have almost no effect on following buses. The lag phase was followed by a transient phase in which travel times significantly increased during a relatively short period of time. A stabilized phase then followed in which travel times fluctuated about a relatively high value.

An upper limit of 16 min was observed during the test periods in both the eastbound and westbound directions. This upper limit is associated with the capacity at the inflow control gates (although 120, 138, and 150 buses/h was attempted at the control gates, only about 120 buses/h and 100-115 buses/h passed through the eastbound and westbound control gates, respectively) and the traffic conditions downstream of the exiting gate (large bus volumes were not maintained past the control gates).

Because of these characteristics, bus volumes, speeds, and dwell times were analyzed during fixed time periods that began about 10 min after the start of each test period

Table 1. Bus volume eastbound.

		Location								
Analysis Period	Item	Kekaulike	Maunakea	Smith	Nuuanu	Bethel	Fort Street Mall	Bishop	Alakea	Richards
10:00-10:30 a.m.	Number of buses[a]	28	28	28	28	30	30	30	28	28
	Time (min)	27.90	27.13	27.16	27.68	27.32	27.44	26.21	27.27	26.35
	Volume (buses/h)	60	62	62	61	66	66	69	62	64
10:45-11:00 a.m.	Number of buses[a]	27	27	27	27	28	28	28	26	26
	Time (min)	13.67	14.25	14.90	18.61	19.41	20.98	20.26	20.61	20.20
	Volume (buses/h)	119	114	109	87	87	80	83	76	77
11:15-11:30 a.m.	Number of buses[a]	29	29	29	29	30	30	30	29	29
	Time (min)	14.24	17.77	17.79	16.33	16.25	16.86	17.21	17.98	17.97
	Volume (buses/h)	122	98	98	107	111	107	104	97	97
12:10-12:24 p.m.	Number of buses[a]	28	28	28	28	29	29	29	28	28
	Time (min)	13.72	15.47	15.93	16.37	15.95	15.57	17.15	16.84	17.16
	Volume (buses/h)	122	109	105	103	109	112	101	100	98

[a] A difference in the number of buses results partially from buses entering and departing the system at points other than the control points.

Table 2. Bus volume westbound.

		Location									
Analysis Period	Item	Richards	Alakea	Bishop	Fort Street Mall	Bethel	Nuuanu	Smith	Maunakea	Kekaulike	River
10:00-10:30 a.m.	Number of buses[a]	28	28	31	31	31	31	31	31	31	31
	Time (min)	27.95	27.50	28.47	28.23	27.38	27.53	27.77	27.53	27.64	27.93
	Volume (buses/h)	60	61	65	66	68	68	67	68	67	67
10:45-11:00 a.m.	Number of buses[a]	25	25	27	27	27	27	27	27	27	27
	Time (min)	13.43	14.07	15.93	17.07	16.68	16.81	17.01	15.68	15.62	15.77
	Volume (buses/h)	112	107	102	95	97	96	95	103	104	103
11:15-11:30 a.m.	Number of buses[a]	26	26	27	27	27	27	27	27	27	27
	Time (min)	15.88	16.50	16.71	17.43	16.68	16.18	15.69	15.87	15.79	15.94
	Volume (buses/h)	98	95	97	93	97	100	103	102	103	102
12:10-12:24 p.m.	Number of buses[a]	21	21	22	22	22	22	22	22	6[b]	22
	Time (min)	11.08	14.03	15.22	14.46	12.33	12.62	12.21	12.30	3.13[b]	11.90
	Volume (buses/h)	114	90	87	91	107	105	108	107	115[b]	111

[a] A difference in the number of buses results partially from buses entering and departing the system at points other than the control points. [b] Incomplete data.

Table 3. Bus speed eastbound.

	Bus Speed (km/h)						
Analysis Period	Kekaulike-Smith	Smith-Nuuanu	Nuuanu-Bethel	Bethel-Fort Street Mall	Fort Street Mall-Bishop	Bishop-Richards	System (without entry delay)
10:00-10:30 a.m.	10.8	15.6	4.8	20.1	13.3	11.7	9.3
10:45-11:00 a.m.	7.9	4.8	2.6	4.5	4.0	9.2	4.7
11:15-11:30 a.m.	2.4	2.4	2.4	4.0	4.0	7.7	3.2
12:10-12:24 p.m.	3.0	2.3	2.4	5.6	4.7	7.6	3.5

Note: 1 km = 0.62 mile.

Table 4. Bus speed westbound.

	Bus Speed (km/h)								
Analysis Period	Richards-Alakea	Alakea-Bishop	Bishop-Fort Street Mall	Fort Street Mall-Bethel	Bethel-Nuuanu	Nuuanu-Smith	Smith-Maunakea	Maunakea-Kekaulike	Kekaulike River	System (without entry delay)
10:00-10:30 a.m.	10.3	9.7	9.0	6.9	22.4	13.2	16.6	15.1	6.8	8.5
10:45-11:00 a.m.	1.9	8.7	7.6	3.5	12.9	7.2	11.9	9.8	6.8	4.0
11:15-11:30 a.m.	1.8	3.7	5.3	6.1	18.5	8.7	13.0	15.3	10.0	4.2
12:10-12:24 p.m.	1.8	2.1	4.2	7.4	22.2	9.8	13.5	9.7	7.7	3.7

Note: 1 km = 0.62 mile.

(corresponding roughly to the stabilized phase). These time periods are defined as the analysis periods.

Bus volumes were determined by dividing the number of buses by the time interval between the first and last bus of each analysis period. Bus volumes in the eastbound direction were calculated for the following intersections on Hotel Street (see Table 1): Kekaulike, Maunakea, Smith, Nuuanu, Bethel, Fort Street Mall, Bishop, Alakea, and Richards. It should be noted that one or two regularly scheduled buses entered Hotel Street at Nuuanu during the analysis periods and were considered in the computations.

There was no problem, with respect to eastbound bus volumes, with a flow rate of 60 buses/h. However, when flow rates of 120, 138, and 150 buses/h were attempted through the control gate, only about 120 buses/h could actually pass this intersection. Flow rates through the other intersections were lower than those past the control gate.

Bus volumes dropped immediately as buses approached Smith, where a nearside bus stop is located. Bus volumes then decreased again at Bishop, Alakea, and Richards. The shortest green time—32 s—occurred at Bishop; a major midblock bus stop is located between Alakea and Richards.

Bus volumes in the westbound direction during each test period were calculated at the following locations (see Table 2): Richards, Alakea, Bishop, Fort Street Mall, Bethel, Nuuanu, Smith, Maunakea, Kekaulike, and River. One regularly scheduled bus route entered Hotel Street at Alakea, thus bypassing the control point. This resulted in an increase of as much as 7 buses/h in the number of buses in the system west of the Alakea intersection in each test period. Again, there was no problem in the westbound direction with a flow rate of 60 buses/h.

The maximum service rate at the Richards control gate ranged between 100 and 115 buses/h. The first bottleneck occurred at Alakea, the location of a nearside bus stop. Past Bishop (the intersection with the shortest green time) and Fort Street Mall (location of a major midblock bus stop), bus volumes decreased further. Bus volumes increased past Fort Street Mall through River.

Average bus speeds (space mean speed) were calculated for links (between intersections) and the system (from the control gate to the end of the system). Delays that resulted when a bus was unable to leave the control gate on schedule were not included in the analysis. All other delays were included in calculations of travel time and speed.

In the eastbound direction, system bus speeds from the control gate to Richards ranged from 3.2 to 4.7 km/h (2.0 to 2.9 mph) during the large flow rates, as given in Table 3. This is a decrease from 9.3 km/h (5.8 mph) during the 60-buses/h analysis period. Average link speeds decreased as buses approached Bethel and increased as buses left Bethel. A major nearside bus stop is located at this intersection.

Table 5. Selected performance characteristics of buses at bus stops.

Bus Stop	Test Flow Rate (buses/h)	Avg Link Speed (km/h)	Avg Bus Volume (buses/h)	Avg Dwell Time (s)	Upstream Green Time (s)
Near side					
Smith	120	7.9	109	12	48
Bethel	120	2.6	87	23	48
Alakea	120	1.9	107	11	40
Bethel	120	3.5	97	15	48
River	120	6.8	103	16	44
Smith	138	2.4	98	13	48
Bethel	138	2.4	111	21	48
Alakea	138	1.8	95	11	40
Bethel	138	6.1	97	13	48
River	138	10.0	102	16	44
Smith	150	2.9	105	12	48
Bethel	150	2.4	109	22	48
Alakea	150	1.8	90	11	40
Bethel	150	7.4	107	14	48
River	150	7.7	111	-[a]	44
Average	150	4.5	102	15.0	
Midblock					
Alakea	120	9.2	77	23	-[b]
Union	120	7.6	95	27	52
Alakea	138	7.7	97	19	-[b]
Union	138	5.3	93	25	52
Alakea	150	7.6	98	21	-[b]
Union	150	4.2	91	32	52
Average		6.9	92	24.5	
Far side					
Nuuanu	120	7.2	95	12	48
Maunakea	120	9.8	104	13	-[b]
Nuuanu	138	8.7	103	10	48
Maunakea	138	15.3	103	12	-[b]
Nuuanu	150	9.8	108	9	48
Maunakea	150	9.7	115	13	-[b]
Average		10.1	105	11.5	

Note: 1 km = 0.62 mile.
[a]Not available. [b]No traffic signal.

Table 6. Composition of traffic eastbound.

Site	Type of Vehicle	60 Buses per Hour[a] Number	60 Buses per Hour[a] Percent	120 Buses per Hour[a] Number	120 Buses per Hour[a] Percent	138 Buses per Hour[a] Number	138 Buses per Hour[a] Percent	150 Buses per Hour[a] Number	150 Buses per Hour[a] Percent
Maunakea	Bus	60	40	118	46	123	58	128	50
	Commercial	14	10	22	8	9	4	16	6
	Automobile or taxi	74	50	120	46	81	38	112	44
	Total	148		260		213		256	
Fort Street Mall	Bus	65	23	96	38	110	39	110	41
	Commercial	36	13	28	11	29	10	18	7
	Automobile or taxi	182	64	126	51	144	51	142	52
	Total	283		250		283		270	
Alakea	Bus	65	18	86	22	101	22	96	25
	Commercial	50	14	38	10	19	4	26	7
	Automobile or taxi	254	68	272	68	343	74	254	68
	Total	369		396		463		376	

Composition by Test Period (vehicles/h).

[a]Represents the bus flow attempted at the control gates and is used solely to identify a specific time or test period. As stated earlier, the control gates could handle only 100-120 buses/h.

Table 7. Composition of traffic westbound.

Site	Type of Vehicle	60 Buses per Hour[a] Number	60 Buses per Hour[a] Percent	120 Buses per Hour[a] Number	120 Buses per Hour[a] Percent	138 Buses per Hour[a] Number	138 Buses per Hour[a] Percent	150 Buses per Hour[a] Number	150 Buses per Hour[a] Percent
Alakea	Bus	70	23	120	38	96	34	102	39
	Commercial	38	13	34	11	12	4	6	2
	Automobile or taxi	190	64	166	51	174	62	156	59
	Total	298		320		280		264	
Fort Street Mall	Bus	65	24	106	34	103	36	82	34
	Commercial	26	10	34	11	34	12	17	7
	Automobile or taxi	182	66	173	55	149	52	142	59
	Total	273		313		286		241	
Maunakea	Bus	67	20	103	39	108	29	111	42
	Commercial	48	14	31	12	42	11	15	6
	Automobile or taxi	221	66	132	49	219	60	138	52
	Total	336		266		369		264	

Composition by Test Period (vehicles/h)

[a]Represents the bus flow attempted at the control gates and is used solely to identify a specific time or test period. As stated earlier, the control gates could handle only 100-120 buses/h.

Table 8. Occurrences of bus noise levels in excess of the 92-dB(A) state standard.

	Fort Street Mall							
	North		South		Bishop Street (north)		Bethel Street (south)	
Time Period	Sound Level [dB(A)]	No. of Exceedences	Sound Level [dB(A)]	No. of Exceedences	Sound Level [dB(A)]	No. of Exceedences	Sound Level [dB(A)]	No. of Exceedences
9:30-10:00 a.m.	89	0	90	0	91	0	-[a]	-[a]
10:00-10:30 a.m.	80	0	89	0	92	0	89	0
10:30-11:00 a.m.	-[a]	-[a]	90	0	93	1	86	0
11:00-11:30 a.m.	89	0	86	0	94	1	86	0
11:30 a.m.-12:00 n.	-[a]	-[a]	90	0	90	0	87	0
12:00 n.-12:30 p.m.	92	0	89	0	94	2	89	0

[a]Not available.

In the westbound direction, bus speeds through the system ranged from 3.7 to 4.2 km/h (2.3 to 2.6 mph) during the large flow rates, as given in Table 4. This is a decrease from 8.5 km/h (5.3 mph) during the 60-buses/h analysis period. Average link speeds were low at the beginning of the system and peaked at the Bethel-Nuuanu link.

Bus dwell times were averaged for each of the nine bus stops located at the test site (three on the south side of Hotel Street and six on the north side). Average dwell times are presented for regularly scheduled buses and test buses during the four analysis periods.

Only three eastbound bus stops were involved in the study of bus dwell time: (a) Hotel at Smith, (b) Hotel at Bethel, and (c) Hotel at Alakea. There are two major bus stops in the eastbound direction: Bethel and Alakea. During large flow rates, dwell time averaged between 21 and 23 s at Bethel, a nearside bus stop, and between 19 and 23 s at Alakea, a midblock bus stop. At Smith, a minor nearside bus stop, dwell times averaged 12 to 13 s during large flow rates.

Six westbound bus stops were involved in the study of dwell time: (a) Hotel at Alakea, (b) Hotel at Union Mall, (c) Hotel at Bethel, (d) Hotel at Nuuanu, (e) Hotel at Maunakea, and (f) Hotel at River. Union Mall bus stop is the major bus stop in the westbound direction. Average dwell time at this midblock bus stop ranged between 25 and 32 s. Dwell times at other nearside bus stops—Alakea, Bethel, and River—and at farside bus stops—Nuuanu and Maunakea—averaged less than 17 s during the analysis periods.

During the bus demonstration, it was observed that bus stops were a major bottleneck. Table 5 gives selected performance characteristics of buses during the last three analysis periods. These data are categorized for nearside, midblock, and farside bus stops.

The data indicate that, of the three types of bus stops, nearside bus stops at signalized intersections had the greatest adverse impact on bus speeds. The average link speed with nearside bus stops was 4.5 km/h (2.8 mph) compared with 6.9 and 10.1 km/h (4.3 and 6.3 mph) on links with midblock and farside bus stops, respectively.

When links with bus stops are considered, it appears that average dwell times or green times did not have a significant impact on average link speed. There was very little correlation between average link speed and average dwell time ($r = 0.1$) or green time ($r = 0.3$).

Bus volumes on links with nearside bus stops ranged from 87 to 111 buses/h and averaged 102 buses/h. On midblock links, bus volumes ranged from 77 to 98 buses/h and averaged 92 buses/h. Links with farside bus stops had bus volumes that ranged from 95 to 115 buses/h and averaged 105 buses/h.

Although the range and average bus volume on links with nearside bus stops were similar to those on links with farside bus stops, average link speed on links with nearside stops was 125 percent lower.

Average link speeds were 54 percent lower on links with nearside bus stops than on links with midblock bus stops, but average bus volumes were 10 buses/h lower on links with midblock bus stops. To determine whether this lower bus volume was related to higher link speeds, selected characteristics of bus performance were tabulated for links with nearside bus stops that had bus volumes within the same range as the links with midblock bus stops. The table below, which gives this information, uses a bus volume of 77 to 98 buses/h and indicates that average link speeds were still lower (by 126 percent) on links with nearside bus stops (1 km = 0.62 mile):

Bus Stop	Avg Link Speed (km/h)	Avg Bus Volume (buses/h)	Avg Dwell Time (s)
Bethel	2.6	87	23
Alakea	1.8	90	11
Alakea	1.8	95	11
Bethel	6.1	97	13
Bethel	3.5	97	15
Smith	2.4	98	13
Average	3.1	94	14.3

Since other variables that could have affected bus speeds, such as signal timing or block length, were not analyzed, the conclusions presented in this section require further analysis.

Other Vehicle Movements

The following types of vehicle surveys were conducted during the bus demonstration: (a) 24-h traffic count, (b) queue length on side streets, (c) traffic composition, and (d) speed and delay.

Twenty-four-hour traffic counts at selected sites along Hotel Street indicated no significant differences in traffic volumes during the 24-h period and the 2.5-h test period for the test day and other previous days.

Traffic queue counts for four side streets were obtained during the experiment. Queues that approached the intersections at the end of the red phase were counted at Nuuanu, Bethel, Bishop, and Alakea. Nuuanu was the only surveyed roadway that reported an occasional fully loaded signal cycle during the study.

Surveys of traffic composition were taken at three sites for both eastbound and westbound directions along Hotel Street. Surveyors located at these sites recorded the number of city buses, other buses, commercial vehicles, automobiles, and taxis in 5-min intervals. Motorcycles and bicycles were not included in the totals.

Traffic composition was determined for three sites located in the eastbound direction of Hotel Street just west of (a) Maunakea, (b) Fort Street Mall, and (c) Alakea (see Table 6). Total traffic counts, from 10:00 a.m. to 12:30 p.m, for city buses, commercial vehicles, and automobiles and taxis ranged from 976 vehicles at Alakea to 551 vehicles at Maunakea. At Fort Street Mall, 679 vehicles were recorded. During this period, the largest category of vehicles recorded was automobiles and taxis, which made up between 48 and 69 percent of the vehicles recorded at the three sites. This was followed by city buses (22 to 43 percent) and commercial vehicles (9 to 11 percent).

Traffic composition was also determined for the three sites located in the westbound direction of Hotel Street just west of (a) Alakea, (b) Fort Street Mall, and (c) Maunakea (see Table 7). During the study period, the totals of city buses, commercial vehicles, and automobiles and taxis at these sites were about the same. Vehicle counts ranged from 701 to 757 vehicles. During this period, automobiles and taxis made up 59 to 60 percent of the recorded traffic, the largest of the three categories at the three sites. This was followed by city buses (30 to 32 percent) and commercial vehicles (9 to 10 percent).

A total of 35 speed and delay runs were made in the study area from 10:00 a.m. to 12:30 p.m. Each of the five roadways surveyed—Hotel, King, Beretania, Bishop, and Alakea—were divided into three links of reasonably uniform physical and traffic characteristics. Speed and percentage of travel time attributable to delays were calculated for each link of each test run. Delays resulted from traffic signals, traffic backups, pedestrians, buses, and turning vehicles.

Six automobile runs were made on Hotel Street in the eastbound direction. The average system speed during the 60-buses/h analysis period was 10.4 km/h (6.4 mph). During the periods of greater bus flows, system automobile speeds were 7.8 km/h (4.8 mph), 6.1 km/h (3.8 mph), and 6.6 km/h (4.1 mph), respectively.

Four automobile speed and delay runs were made on Hotel Street in the westbound direction. The average system speed during the 60-buses/h analysis period was 17.7 km/h (11.0 mph). During the periods of greater bus flows, system automobile speeds were 10.3 km/h (6.2 mph), 7.0 km/h (4.3 mph), and 7.2 km/h (4.5 mph), respectively.

Pedestrian Movements

During the 2.5-h test period, a total of 12 939 pedestrians crossed Hotel Street at three intersections: Bishop, Bethel, and Fort Street Mall. During the pedestrian peak-hour period, between 11:30 a.m. and 12:30 p.m., 6727 pedestrians crossed Hotel Street at the three intersections.

The heaviest pedestrian traffic occurred at the intersection of Hotel Street and Fort Street Mall. A total of 6394 pedestrians used this intersection during the test period, and the average flow rate was 2558 pedestrians/h. The 15-min pedestrian counts ranged from a low of 459 pedestrians during the earlier portion of the test period (or a flow rate of 1836 pedestrians/h) to a high of 1014 pedestrians during the latter portion of the test period (or a flow rate of 4056 pedestrians/h).

Environmental Impacts

Existing standards for the state of Hawaii were used to evaluate the results of noise and air pollution measurements taken during the experiment.

The noise level of heavy vehicles on any traffic way with a posted speed limit of 56 km/h (35 mph) or less is not to exceed 92 dB(A) [fast meter response measured at 6 m (20 ft) from the centerline] during daytime hours (6:00 a.m to 6:00 p.m.) (5). Four noise analyzers monitored noise levels 9 m (30 ft) from the center of Hotel Street.

All noise levels recorded at Fort Street Mall (north and south) and Bethel (south) were within the standard. However, as given in Table 8, noise levels at Bishop exceeded 92 dB(A) four times during the 3-h test period. The maximum noise level recorded was 94 dB(A). The high noise level at this site may be attributable to the fact that the noise meter was located within 6 m (20 ft) of the noise source and approximately 1.5 m (5 ft) of a concrete structure, an effective sound-reflecting surface.

Average levels of carbon monoxide (CO) and sulfur dioxide (SO_2) did not exceed Hawaii standards. The table below gives 1-h average levels of CO for Fort Street Mall south [the state CO standard is 10 mg/m^3 (8.7 ppm)] (1 mg/m^3 = 0.87 ppm):

Time Period	CO (mg/m³)	Time Period	CO (mg/m³)
9:30-10:00 a.m.	5.75	11:00-11:30 a.m.	5.75
10:00-10:30 a.m.	5.2	11:30 a.m.-12:00 n.	4.0
10:30-11:00 a.m.	6.3	12:00 n.-12:30 p.m.	4.6

The table below gives the 3-h average levels of SO_2 observed (the state SO_2 standard is 400 $\mu g/m^3$):

Time Period	SO_2 ($\mu g/m^3$) Fort Street Mall	Bishop Street
10:00-11:00 a.m.	2.3	3.3
11:00 a.m.-12:00 n.	2.3	11.4
12:00 n.-12:40 p.m.	3.4	12.2

CONCLUSIONS

A brief discussion of the conclusions of the study is presented here. A more detailed discussion is available elsewhere (4).

Bus Movements

The Hotel Street bus demonstration indicated a bus capacity of 95-100 buses/h. This falls within the highest observed volume range (90-120 buses/h) on single-lane, downtown streets with on-line bus stops (1, 3). However, average bus speeds through Hotel Street were 3-5 km/h (2-3 mph) rather than the 8-16 km/h (5-10 mph) associated with the highest observed volume in other areas.

In the eastbound direction, the limiting bus volume passed through Hotel Street at Richards. Data at this intersection indicated that bus capacity in the eastbound direction was about 100 buses/h. A major bottleneck was located on the Nuuanu-Bethel link. The average link speed was about 2.4 km/h (1.5 mph) during the heavier flow rates, the lowest through the system in the eastbound direction. The Smith-Nuuanu link, the immediate upstream link, also recorded below-average system speeds during the 24- and 26-s-headway test periods.

In the westbound direction, minimum bus volumes passed Hotel Street at Fort Street Mall. Data at this intersection indicated that westbound bus capacity on Hotel Street is about 95 buses/h. The first major bottleneck identified was located on the Richards-Alakea link. Travel speeds on this link averaged 1.8 km/h (1.1 mph) during the heavier flow rates, the lowest through the system. Average link speeds were also below the average system speed on the Alakea-Bishop link during the attempted 26- and 24-s-headway test periods.

During the demonstration, it was observed that bus stops were a major bottleneck. Average speeds on links with nearside bus stops had the lowest link speeds compared with links with midblock or farside bus stops.

In 1967, the Institute of Traffic Engineers recommended the placement of nearside bus stops at signalized intersections where transit movement (assuming low bus volumes) is critical and parking and traffic are not (6). However, from the limited observations made in this study, it appears that nearside bus stops at signalized intersections are not superior to midblock or farside bus stops with respect to speed or travel time for heavy rates of bus flow. However, further analysis must be done to verify this conclusion.

Other Vehicle Movements

Twenty-four-hour traffic counts at selected sites along Hotel Street indicated no significant differences in traffic volume during the 24-h period and the 2.5-h test period for the test day and other previous days. Travel time studies showed that, although overall automobile speeds were higher than bus speeds—usually by almost 100 percent—automobile speeds were adversely affected by high bus volumes. Checks of queue length on side streets during the experiments showed no problem.

Pedestrian Movements

The intersection at Hotel Street and Fort Street Mall served the largest pedestrian demand: 6394 pedestrians between 10:00 a.m. and 12:30 p.m. A signalized crosswalk is located at this intersection to satisfy this large pedestrian demand. This additional pedestrian signal adversely affected bus movements on Hotel Street. Since at other locations along Hotel Street pedestrians crossed Hotel Street with cross vehicle traffic and no significant number of jaywalkers were recorded, pedestrian movements were not a problem at other intersections.

Environmental Impacts

No air quality problems were recorded by the Hawaii Department of Health. Wind speed and direction were 33 km/h (20 mph) east-northeast with gusts up to 46 km/h (29 mph) during the day.

Four occurrences of noise levels exceeding the state standard of 92 dB(A) were recorded along Hotel Street at Bishop. This probably resulted from the sound-reflecting background at that location since the same buses did not exceed the state standard at other locations.

It should be noted that current state noise standards for vehicles are defined for distances between 6 and 15 m (20 and 50 ft) from the source. However, bus noise on Hotel Street will have its greatest impact on pedestrians located on the adjacent sidewalk. Since sidewalks are located at the edge of the roadway and are approximately 2.4 m (8 ft) wide, noise levels experienced by pedestrians could be higher than 92 dB(A). This is an important factor in considering high bus volumes on Hotel Street and the application of existing state noise standards to CBD areas.

ACKNOWLEDGMENT

We wish to thank the following agencies and organizations for their assistance in this study: the Hawaii Department of Health, the Health Department (Emergency-Health Services Division) of the city and county of Honolulu, the Honolulu Fire Department and Police Department, MTL, and the Oahu Civil Defense Agency. We would also like to acknowledge Peter Ho and Mark Kaneshiro, who were involved in the development of this study. Alan Kim drafted the figure and the original tables.

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**K. Hayashida was with the Honolulu Department of Transportation Services when this study was performed.*

Discussion

Herbert S. Levinson, Wilbur Smith and Associates

The paper by Hayashida, Fujita, Hamayasu, and Lum provides a much needed addition to the state of the art on bus capacity. It is indeed gratifying to find research based on actual field tests. The findings are generally consistent with the range of values reported for other downtown areas. They show what bus capacities can be realized without the "leapfrogging" of buses.

The authors found that bus speeds declined as flow rates increased. Westbound, speeds dropped from about 8.5 km/h for a flow of 60 buses/h to about 4.0 km/h for a flow of 100-120 buses/h; eastbound, speeds declined from about 9.3 to 4.0 km/h for the same increases in bus flow rates. More detailed analysis of these relations would be desirable to allow a level-of-service concept to be introduced into the analysis.

The paper implies, but does not clearly state, that passengers boarding and alighting from regularly scheduled buses at major load points limit the capacity of the system (as at Bethel and Alakea eastbound and Fort Street Mall westbound). More analysis and interpretation of the interrelationship between regularly scheduled and test buses are desirable.

Important information is lacking in several areas:

1. It is not clear how many automobiles traveled in the bus lane during each test period. These automobiles occupy a portion of the green time that would otherwise be available for buses.
2. The effects of multiple use of bus stops and use of multiple berths are not clearly specified. Data on the amount of bus queuing at bus stops along Hotel Street seem to be lacking (the authors indicate that two buses are allowed to load and unload simultaneously at each stop).
3. The effects of varying the dwell times of test buses are not indicated.

The analysis would be strengthened by a fuller discussion of certain operational recommendations that emerged from the research. Can capacity be increased by providing only farside stops? by dispersing loading points? by lengthening bus stops? Perhaps answers to questions such as these can form a logical extension to this important and timely research.

Ann Muzyka, Transportation Systems Center, U.S. Department of Transportation

The Honolulu Department of Transportation Services is to be commended for undertaking the experiment described by the authors. Data collected in the field under operating conditions are extremely valuable; they are needed to keep analysis in touch with reality. The amount and variety of data collected are impressive. The negligible influence of increased bus volumes on pollution and noise levels is significant. This study will be valuable for other researchers in planning similar experiments as well as in developing theories and procedures for estimating capacity. Such analytical techniques must account for the facts developed in this study.

It is clearly stated that the objective of this experiment was to determine bus capacity under existing conditions. Therefore, the traffic signal timings, traffic volumes, and bus stop locations remained constant. The parameters of traffic signal timings, especially the offset pattern, are very important in filtering the movement of vehicles (7). A useful future study would be an analysis and field experiment to determine the influence of traffic signal timings on street capacity for buses. An appropriate performance measure would be person throughput rather than vehicle throughput.

I understand the increase in bus volumes is contemplated to improve the level of service for current demand and not to accommodate projected increases in demand. Additional no-passenger buses used in the experiment were given 15-s dwell times at each bus stop to simulate passenger loading, and these data were not separated from the dwell times of buses that carried passengers. It would have been useful to separate service times for the two types of buses. In addition, an explanation of why the 15-s dwell time was chosen for all no-passenger buses would be interesting.

In summary, the paper is well written and extremely interesting. This study is significant for its direct approach of metering bus-flow rates until saturation is reached, a common practice in simulation studies. It is a valuable source of field data, which provide the acid test for relevant theories and models. The demonstration procedures and results will be useful to all planners of similar experiments.

REFERENCE

7. E.B. Lieberman, A. Muzyka, and D. Schmeider. Bus Priority Signal Control: Simulation Analysis of Two Strategies. TRB, Transportation Research Record 663, 1978, pp. 26-28.

Analysis of Intersection Capacity and Level of Service by Simulation

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A procedure for applying computer simulation in evaluating the capacity and level of service of single, unsignalized intersections is presented. The process may also be used to study these features at signalized intersections. The TEXAS model for intersection traffic is especially suitable for this purpose because it (a) uses a detailed description of intersection geometry, (b) incorporates as many as 5 driver and 15 vehicle classes in the traffic stream, (c) simulates the behavior of each individually characterized driver-vehicle unit as it responds to its static and dynamic surroundings, and (d) presents summary statistics about the performance of traffic and traffic control devices at the end of any selected period of time. Capacity, which is the maximum traffic volume that can be accommodated under prevailing conditions on an intersection approach or by the whole intersection, can be determined through a successive approximation technique by using a few runs of the model. The level of service of an intersection operating under a specified form of control and carrying a given traffic volume can be defined in terms of recommended quantitative indicators such as average delay, percentage of vehicles required to stop, and percentage of vehicles required to slow to less than 16 km/h (10 mph). Four cases in which the model can be used to determine the capacity and level of service of unsignalized intersections are presented.

The maximum volume of traffic that can be handled by a road or street network is frequently limited to that which can flow through a single intersection. Capacity analysis of road segments that include intersections thus involves two basic steps:

1. Critical, or bottleneck, intersections must be identified and their capacity determined and
2. The overall traffic-carrying capability of the road section can then be appraised.

Thus, a practical, effective means is needed for evaluating the capacity of a single intersection operating under any given form of traffic control.

The capacity of an intersection is defined as the maximum number of vehicles that can pass through the intersection during a given period of time under prevailing roadway, environmental, and traffic conditions (1, p. 129). Capacity is not a constant quantity but depends on a number of factors, some of which are static (e.g., intersection geometry and traffic control devices) and others of which are dynamic (e.g., moving vehicles and pedestrians). At a particular time, the maximum flow on an intersection approach, or through the intersection as a whole, might be considerably different than at another time because of different traffic patterns or other dynamic factors. Analysis of intersection capacity involves evaluating the combined effects of both static and dynamic influences and defining the maximum volume that can be accommodated on each intersection approach under the stated conditions without specific regard for the degree of satisfaction that will be experienced by the driver. When it operates at capacity, an intersection usually provides relatively poor service from the viewpoint of the user.

Level of service is a phrase used by transportation engineers to describe the subjective appraisal that a representative driver will give to the quality of traffic flow provided by an intersection approach. Associated with each service level is a service volume—the maximum volume the intersection can accommodate while providing the specified level of service. If satisfied with the manner in which traffic moves through the intersection, the driver will say that a high level of service is provided; if dissatisfied, the driver will indicate a low level of service. Experience has shown, however, that what is judged to be excellent service under one set of circumstances at one location may well be described qualitatively as poor service in a different situation. Quantitative indicators that differentiate various levels of service under defined conditions are needed to make communication among transportation professionals easier and to achieve consistency in intersection evaluation and design.

The methods currently available for analyzing intersection capacity and levels of service are generally empirical, probabilistic, or based on sample observations. In the empirical methods, historical experience and analysis are usually reduced to charts, tables, and adjustment factors. Probabilistic methods use statistical distributions to represent traffic characteristics such as headway, spacing, and speed. Expected interactions among traffic streams at the intersection are then computed and presented as graphs or formulas for capacity. Observation methods involve field sampling and forecasting. Time-lapse photography has been used successfully to record traffic movements at representative intersections; then, data from the pictures have been analyzed and reduced to formulas for capacity. These methods have usually been applied to capacity evaluation of signalized intersections on a macroscopic scale.

No means other than direct observation has been available for studying, on a microscopic scale, intersection capacity as it is affected by the behavior of individually characterized driver-vehicle units operating in the partly static, partly dynamic intersection environment. Recent advances in digital computer technology now make this possible, however, through simulation. The expected interaction among the four primary elements of intersection traffic flow—the driver, the vehicle, the roadway, and the traffic control—can be analyzed by computer simulation in considerable detail and in a highly compressed time frame. A particularly suitable simulation package for this purpose is the traffic experimental and analytical simulation (TEXAS) model for intersection traffic (2-6), which was developed at the Center for Highway Research at the University of Texas at Austin especially for analyzing traffic performance at single, multileg, mixed-traffic intersections that operate either without control devices or with any conventional sign or signal control scheme.

This paper describes how results of simulation with the TEXAS model have been used as the basis for selecting suitable quantitative indicators of level of service and for developing a procedure for determining the capacity and level of service of unsignalized intersections. Only a few runs of the model are required to evaluate the expected performance of the geometric configuration, control scheme, and traffic pattern of any selected intersection.

OPERATION OF THE MODEL

The TEXAS model accomplishes a microscopic, step-through simulation of traffic flow at a single intersection. It is a deterministic model for the most part in that none of the response decisions are based on probability. Traffic input to the modeled intersection approaches is generated on a stochastic basis from descriptive information provided by the user. Arrival headways are generated by a preprocessor in the computer program as random variates of a user-selected probability distribution function. Then, when precise criteria required for a particular driver-vehicle response are satisfied, a programmed action is carried out. Each driver-vehicle unit in the intersection area is examined sequentially during a short time interval (e.g., 0.5 s) and advanced to its next position.

The simulated intersection system is assumed to attain a steady-state condition after a specified start-up time. During start-up time, all movements are simulated but no performance statistics are gathered. After that, all traffic and control activities are simulated, and statistics are accumulated as each vehicle logs out of the system at the end of the outbound lane. Summary statistics are reported in a tabular form at the end of the specified simulation time.

On request, a large variety of information concerning the results of simulation can be printed, punched on cards, or shown on a graphics display screen. The data can be produced for each traffic movement separately, according to approach, or they can be summarized for the whole intersection.

The items of output that have been used in this paper for quantifying capacity and level of service at unsignalized intersections are (a) total intersection volume, (b) percentage of vehicles required to stop, (c) percentage of vehicles required to slow to less than 16 km/h (10 mph), (d) average queue delay, and (e) average stopped delay.

LEVEL-OF-SERVICE INDICATORS

The level of service at intersections depends on the manner in which traffic flows through the intersection. At signalized intersections, load factor is widely accepted as a performance indicator for level of service (1). Load factor is defined as the ratio of the total number of fully used green signal intervals in a series of signal cycles to the total number of green intervals for that approach during the same period. Load factor is easy to measure in the field since all that is required is a count of the green phases during which vehicles are present throughout the phase and the total number of green phases displayed in a selected time period. Load factor is the ratio of these two numbers. Numerical limits of load factor for various levels of service are given below (1):

Level of Service	Traffic Flow	Load Factor
A	Free	0.0
B	Stable	≤0.1
C	Stable	≤0.3
D	Approaching unstable	≤0.7
E	Unstable	≤1.0
F	Forced	–

Even though load factor is used extensively to identify intersection levels of service, it is not an ideal descriptor. Its applicability to signalized intersections is limited, and the break points between the various levels of service have no strong rational basis. A better and more widely applicable means is needed for expressing the quality of intersection performance as perceived in quantitative terms by the user.

Indicators that can be used at intersections with all forms of traffic control are needed to identify the level of service that is provided. The selection of appropriate indicators can be considered from two points of view. The designer prefers indicators that can be measured easily in quantitative terms, whereas the user may prefer more subjective measures of satisfaction. Indicators related to both points of view should be selected for evaluating the performance of intersections. The selected indicators must be easy to measure quantitatively, and the user must be able to relate them to his or her personal satisfaction. If simulation is to be used in capacity analysis, any indicator of level of service should be readily attainable from the simulation model.

The indicators discussed below appear to be appropriate measures of level of service at unsignalized intersections in that they incorporate all of these desired features.

Queue Delay

Queue delay is the time spent by a vehicle at a virtual stop in a queue on an intersection approach. A vehicle can be said to be in a queue when it is within, say, 10 m (33 ft) of another vehicle, or some other object, ahead that requires a stop and when it is stopped or moving less than 3 km/h (2 mph). Queue delay begins when the vehicle joins a queue and ends when the vehicle enters the intersection.

Once a vehicle is in a queue, it is considered to remain in the queue until it enters the intersection even if its speed exceeds 3 km/h (2 mph) as it moves forward in the queue. Queue delay thus includes time spent in moving up in the queue. Since vehicles at unsignalized intersections experience this type of delay, queue delay is an appropriate criterion that may be used to evaluate delay at unsignalized intersections. Queue delay is readily identified by the user as an index of intersection performance since the user prefers to spend a minimum of time waiting in a queue while traveling through an intersection. Since average queue delay is one of the statistics compiled from simulation by the TEXAS model, it is a readily available quantitative factor that may be used as a level-of-service indicator.

In field studies, queue delay can be measured by (a) a count of the number of vehicles in the queue at fixed, periodic time intervals (point sample), (b) the input-output method, (c) a path trace based on a sample of individual vehicles, and (d) time-lapse photography. A special device for recording queue delay by the point-sample technique on a 1-s time basis is described by Lee, Rioux, and Copeland (2).

A recent study by Sutaria and Haynes (7) used the opinions of 310 drivers who had a wide variety of driving experience to evaluate intersection levels of service. Each participant in the study was first asked to rank the following factors according to their relative importance in defining the quality of service provided by an intersection: (a) delay, (b) number of stops, (c) traffic congestion, (d) number of trucks and buses in the traffic stream, and (e) difficulty of changing lanes. Each driver was then shown a series of photographs of a signalized intersection in Dallas that was operating under a variety of traffic conditions or levels of service. A majority of the drivers indicated, both before and after viewing the pictures, that delay was the most important factor in their subjective evaluation of intersection performance.

Percentage of Vehicles Required to Stop

The percentage of vehicles that are required to stop is easy to measure in the field by simply counting all the vehicles that stop and the total traffic volume for a selected period of time. No special equipment is required for these measurements. It is apparent to the driver that the intersection functions more satisfactorily if most vehicles can pass through without having to stop. This parameter is also available in the summary statistics of the TEXAS model. Since percentage of vehicles required to stop is easier to measure than average queue delay, it might be preferable to intersection designers as a level-of-service indicator. It is applicable primarily at uncontrolled and yield-sign-controlled intersections, however, since, at stop-sign-controlled intersections, all vehicles on approaches facing the stop signs are required to stop. An advantage of using this parameter is that the stage at which an uncontrolled or yield-sign-controlled intersection behaves essentially as a stop-sign-controlled intersection can be observed because, at that point, a high percentage of vehicles are required to stop.

Figure 1. Levels of service at four-lane by four-lane, all-way stop-sign-controlled intersection: service volume versus average queue delay.

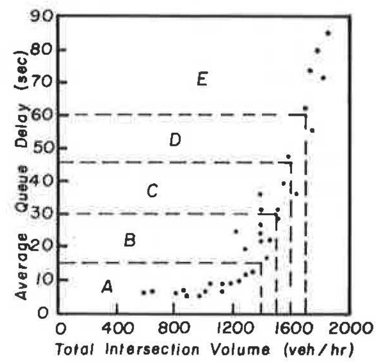


Figure 2. Levels of service at yield-sign-controlled intersection: percentage of vehicles on signed approaches slowing to less than 16 km/h versus average queue delay.

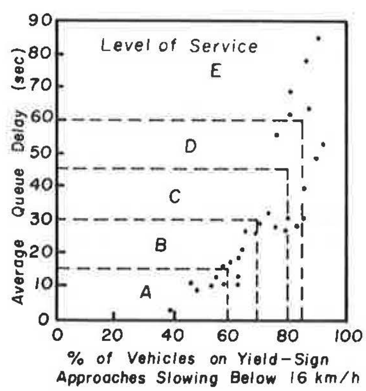
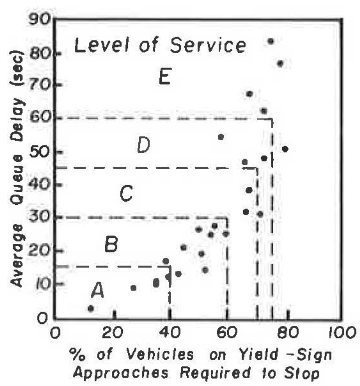


Figure 3. Levels of service at yield-sign-controlled intersection: percentage of vehicles on signed approaches required to stop versus average queue delay.



Percentage of Vehicles Required to Slow to <16 km/h (<10 mph)

The percentage of vehicles that must slow to less than 16 km/h (10 mph) relates directly to driver satisfaction since no driver likes to slow to less than 16 km/h. This indicator is difficult to determine in field studies, however. It can possibly be measured in the field by using time-lapse photography. The TEXAS model allows the user to specify the minimum desirable speed, and then the percentage of simulated vehicles that traveled at less than this speed is computed. Thus, a statistic is available for comparing the performance of various types of unsignalized intersections. A further incentive for considering this indicator is that the 1971 Manual on Uniform Traffic Control Devices (MUTCD) (8, p. 34) states the following:

> The yield sign may be warranted: On a minor road at the entrance to an intersection where it is necessary to assign right-of-way to the major road, but where a stop is not necessary at all times, and where the safe approach speed on the minor road exceeds [16 km/h] 10 miles per hour.

RELATING SELECTED PERFORMANCE INDICATORS TO LEVEL OF SERVICE

Since queue delay can be used as an indicator of level of service for all types of intersection control, a quantitative relation between queue delay and level of service, similar to the one that has been recognized between load factor and level of service, is desired. Once this relation is established, the maximum volume that can be accommodated at each level of service can be determined.

May and Pratt (9) used the results of simulation to correlate average delay with load factor for signalized intersections and thus linked average delay to level of service as described in the 1965 Highway Capacity Manual (HCM) (1). This relation is presented in an analysis by May and Pratt (9, Table 2, p. 47) that defines reasonable and orderly relations between average delay and level of service at signalized intersections. Recent observational work by Sutaria and Haynes (7, Figure 2, p. 111) adds support to this concept and defines quite similar break points for the various levels of service. The results of the work of May and Pratt and Sutaria and Haynes are given below:

Level of Service	Average Individual Delay (seconds per vehicle) May and Pratt	Sutaria and Haynes
A	≤ 15	≤ 12.6
B	≤ 30	≤ 30.1
C	≤ 45	≤ 47.7
D	≤ 60	≤ 65.2
E	> 60	≤ 82.8

Since operational delays for a given level of service should be consistent regardless of the type of control at the intersection, these same values can be used to describe levels of service at unsignalized intersections also.

After making a large number of runs of the TEXAS model for a wide range of traffic demand at a four-lane by four-lane intersection controlled by stop signs on all approaches, a graph was drawn to show total intersection volume versus average queue delay (see Figure 1). Lines that represent average delay at signalized intersections for the various levels of service that are listed by May and Pratt (9, p. 47) and given in the table above have been superimposed on the graph and extended downward from their intercept with the data points to show the related service volume for each level of service. The volume of traffic accommodated at each level of service, or the service volume, can be judged to be reasonable and can be expected to result in the general flow conditions described in the HCM (1). The same type of orderliness in these parameters was found for other lane arrangements.

The MUTCD (8, p. 33) states that one condition that might warrant a multiway stop sign is an average delay of at least 30 s/vehicle during the maximum hour. Since an average delay of 30 s is the upper boundary suggested for level of service B (see the table above), this adds validity to the choice of 30 s as the boundary between levels of service B and C at which intersections normally operate acceptably.

Average queue delay can be used as a measure of level of service for all intersections; however, for uncontrolled and yield-sign-controlled intersections, a more convenient indicator of level of service might be the percentage of vehicles that are required to stop. It is easier to measure the percentage of vehicles stopped than to determine delay. For yield-sign-controlled intersections, another candidate indicator is the percentage of vehicles required to slow to less than 16 km/h (10 mph) since a commonly accepted warrant for that control is that it may be used if the approach speed exceeds 16 km/h.

These two performance indicators—percentage of vehicles required to stop and percentage of vehicles required to slow to less than 16 km/h—may also be related to level of service through simulation studies. To establish these relations, the TEXAS model was run to examine traffic behavior at representative yield-sign-controlled intersections that had various lane arrangements and operated under a wide range of traffic volumes. The average queue delay that resulted from different percentages of vehicles slowing to less than 16 km/h and the average queue delay that resulted from different percentages of vehicles being required to stop were obtained.

The data points in Figure 2 show the percentage of vehicles on yield-sign-controlled approaches that slowed to less than 16 km/h versus average queue delay. Figure 3 shows the percentage of vehicles on the yield-sign-controlled approaches that were required to stop versus average queue delay. In both figures, horizontal lines that define the various levels of service in terms of average queue delay according to May and Pratt (see the preceding table) are superimposed. Each of these lines is extended downward from its intercept with the data points to describe, respectively, the level of service as indicated by the percentage of vehicles on sign-controlled approaches that slowed to less than 16 km/h (10 mph) and the percentage of vehicles on the signed approaches that were required to stop. A summary is given in the table below of the relation among level of service, average queue delay, percentage of vehicles slowing to less than 16 km/h, and percentage of vehicles required to stop for yield-sign-controlled intersections of the two-lane by two-lane and four-lane by four-lane configuration (1 km = 0.62 mile):

Level of Service	Average Queue Delay (s)	Percentage of Vehicles on Yield-Sign-Controlled Approaches: Slowing to < 16 km/h	Required to Stop
A	< 15	< 60	< 40
B	< 30	< 70	< 60
C	< 45	< 80	< 70
D	< 60	< 85	< 75
E	> 60	> 85	> 75

RECOMMENDED PERFORMANCE INDICATORS FOR UNSIGNALIZED INTERSECTIONS

Average queue delay is recommended as the best indicator of level of service for stop-sign-controlled intersections. Average queue delay can be measured in the field by appropriate survey techniques, it can be understood by the user, and it can be simulated by the TEXAS model. The relation between average queue delay and level of service that is applicable to signalized intersections is also recommended for stop-sign-controlled intersections (see the values of May and Pratt in the table on p. 37).

For yield-sign-controlled intersections, the percentages of vehicles on signed approaches that must slow to less than 16 km/h (10 mph) and those that must stop can be considered as good indicators of level of service. The percentage of vehicles that must slow to less than 16 km/h can be determined from simulation or can possibly be measured in the field by using time-lapse photography; this measure can also be understood by the user. In addition, it has been recognized as the basis of a warrant for yield-sign control of intersections. Suggested relations between the percentage of vehicles that slow to less than 16 km/h and the various levels of service are given in the table above. The percentage of vehicles on the signed approaches that must stop can be measured easily in the field, is easily understood by the user, and can be simulated by the TEXAS model. Suggested relations between the percentage of vehicles that must stop and levels of service are also given in the preceding table.

Table 1 is a summary tabulation of recommended performance indicators for various levels of service at each type of unsignalized intersection. The indicator for uncontrolled intersections is consistent with that for yield-sign-controlled intersections. Suggested values for signalized intersections taken from May and Pratt (9) are also included in Table 1.

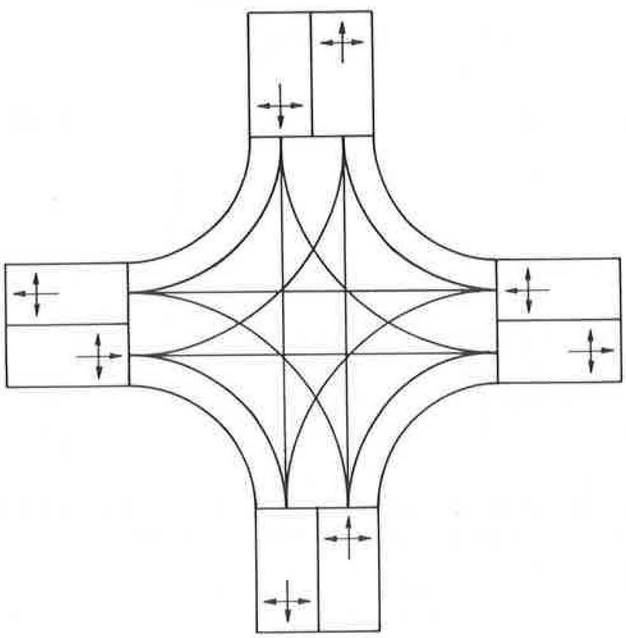
Table 1. Recommended indicators of intersection levels of service.

Type of Intersection Control	Recommended Performance Indicator	Level of Service				
		A	B	C	D	E
Uncontrolled	Percentage of all vehicles that must stop	<40	<60	<70	<75	>75
Yield sign	Percentage of vehicles on sign-controlled approaches that must slow to 16 km/h	<60	<70	<80	<85	>85
	Percentage of vehicles on sign-controlled approaches that must stop	<40	<60	<70	<75	>75
Two-way stop	Average queue delay to vehicles on sign-controlled approaches (s)	<15	<30	<45	<60	>60
All-way stop	Average queue delay to vehicles on all approaches (s)	<15	<30	<45	<60	>60
Signal	Average stopped delay to vehicles on all approaches (s)	<15	<30	<45	<60	>60

Notes: 1 km = 0.62 mile.
Queue delay, the time spent by a vehicle while at a virtual stop in a queue on an intersection approach, is measured from the time the vehicle joins the queue until it enters the intersection and thus includes move-up time. Stopped delay, the time spent by a vehicle while it is actually stopped on an intersection approach, does not include move-up time.

Figure 4. Intersection used for cases 1, 2, and 3.

19TH ST AT CHICON
2-LANE MAJOR/2-LANE MINOR
242-m (800-ft) APPROACHES



SCALE FACTOR IS 1.8 m/cm (15 ft/in)

USE OF THE TEXAS MODEL IN CAPACITY ANALYSIS

An intersection is characterized by its geometry, its type of control, the volume of traffic it accommodates, and the level of service it provides. Generally, if any three of these factors are known, the fourth can be determined. In using the TEXAS model, all known data on geometrics, traffic characteristics, and volume conditions are collected and input to the geometry and driver-vehicle processors and the simulation processor. The summary statistics that are reported from the run are analyzed to provide the required performance information. Four cases in which the TEXAS model can be used to evaluate the performance of an unsignalized intersection are described here.

1. Case 1—Lane configuration, type of control, and service volume accommodated are known; level of service is unknown. The TEXAS model is run with the known geometry and control at the accommodated volume. The value of an appropriate performance indicator is determined from the summary statistics, and level of service is then determined from Table 1.

2. Case 2—Lane configuration, type of control, and level of service are known; the service volume that can be accommodated is unknown. An estimate of the service volume is made. Then the model is run with the geometry, type of control, and estimated volume. The value of the appropriate performance indicator is determined from the summary statistics. The level of service that is provided is determined from Table 1. If this is not the level of service desired, a fresh estimate of the volume is made and the process is repeated until the intersection can be expected to operate at the desired level of service. Usually, two or three runs will be sufficient to estimate the service volume.

3. Case 3—Lane configuration and type of intersection control are known; the level of service provided for different volumes, or the maximum volume that can be accommodated at each level of service, is unknown. The model is run for the known geometry and control at a range of volumes that could be expected to cover all the levels of service. From summary statistics, a graph of volume versus the specific indicator can be drawn, and, by using Table 1 as a guide, a table that links volume to level of service can be constructed and then used to determine the desired information.

4. Case 4—The service volume to be accommodated and the level of service to be provided are known; optimal design (lane configuration and control) is unknown. A lane configuration and a control scheme are chosen. Then the model is run with the desired volume and, from summary statistics and Table 1, the level of service that will be provided is determined. If this level of service is not satisfactory, a fresh choice of geometry and control is made, and the process is repeated until the desired level of service is attained at that volume.

EXAMPLE

To illustrate the use of the TEXAS model in the four cases described above, the following working example is presented. The items to be determined are (a) in case 1, the level of service at which a two-lane by two-lane uncontrolled intersection that accommodates 1600 vehicles/h will operate; (b) in case 2, the maximum volume that can be accommodated by a two-lane by two-lane uncontrolled intersection that operates at level of service B; (c) in case 3, the levels of service at different volumes and the maximum volume that can be accommodated at each level of service by a two-lane by two-lane uncontrolled intersection (analyzed by use of a graph and a table that will be constructed); and (d) in case 4, the optimum lane configuration and type of intersection control to accommodate 1600 vehicles/h while maintaining a level of service A.

Geometry of the Intersection

The intersection is assumed to be a right-angle intersection with four approaches and four exits. For the first three cases, each leg of the intersection has one lane in each direction. The number of lanes for the fourth case will be determined based on the volume to be accommodated and the level of service to be maintained. Each lane is 3 m (10 ft) wide. The influence of the intersection extends 242 m (800 ft) in advance of the intersection on each inbound lane and 121 m (400 ft) beyond the intersection on each outbound lane. The speed limit is 56 km/h (35 mph) on all approaches. There are no restrictions on sight distance. A plot of the intersection used for the example in cases 1, 2, and 3 is shown in Figure 4.

Traffic Data

1. The distribution of traffic on each approach is given below:

Approach	Direction	Percentage of Total Volume
1	Northbound	15
2	Westbound	25
3	Southbound	25
4	Eastbound	35

Table 2. Values supplied by the program for vehicle characteristics.

Vehicle Class	Vehicle Type	Length (m)	Operating Characteristic Factor	Maximum Deceleration (m/s^2)	Maximum Acceleration (m/s^2)	Maximum Velocity (m/s^2)	Minimum Turning Radius (m)	Aggressive Drivers (%)	Average Drivers (%)	Slow Drivers (%)	Percentage in Traffic Stream
1	Small automobile	4.57	100	4.88	2.44	45.73	6.1	30	40	30	20
2	Medium automobile	5.18	110	4.88	2.74	58.54	6.7	35	35	30	32
3	Large automobile	5.79	110	4.88	3.35	60.97	7.32	20	40	40	30
4	Van, minibus	7.62	100	4.88	2.44	45.73	8.54	25	50	25	15
5	Single unit	9.15	85	3.66	2.44	48.78	12.8	40	30	30	0.5
6	Semitrailer	15.24	80	3.66	2.13	48.78	12.19	50	40	10	0.2
7	Full trailer	16.77	75	3.66	1.83	45.73	13.72	50	40	10	0.1
8	Recreational	7.62	90	3.66	1.83	45.73	8.54	20	30	50	0.2
9	Bus	10.67	85	3.66	1.52	38.11	8.54	25	50	25	0.5
10	Sportscar	4.27	115	4.88	4.27	62.5	6.1	50	40	10	1.5

Figure 5. Example of summary statistics used in simulation.

```
TEXAS TRAFFIC AND INTERSECTION SIMULATION PACKAGE - SIMULATION PROCESSOR
*****        NSIM58U - HIGHLAND HILLS - DRIVE AT CIRCLE  *  UNCONTROLLED

SUMMARY STATISTICS FOR ALL APPROACHES

TOTAL DELAY (VEHICLE-SECONDS) -------------------  =  10003.3
NUMBER OF VEHICLES INCURRING TOTAL DELAY --------  =   254
PERCENT OF VEHICLES INCURRING TOTAL DELAY -------  =   100.0
AVERAGE TOTAL DELAY (SECONDS) -------------------  =    39.4
AVERAGE TOTAL DELAY/AVERAGE TRAVEL TIME ---------  =    55.5 PERCENT

QUEUE DELAY (VEHICLE-SECONDS) -------------------  =   7441.0
NUMBER OF VEHICLES INCURRING QUEUE DELAY --------  =   202
PERCENT OF VEHICLES INCURRING QUEUE DELAY -------  =    79.5
AVERAGE QUEUE DELAY (SECONDS) -------------------  =    36.8
AVERAGE QUEUE DELAY/AVERAGE TRAVEL TIME ---------  =    51.9 PERCENT

STOPPED DELAY (VEHICLE-SECONDS) -----------------  =   1674.0
NUMBER OF VEHICLES INCURRING STOPPED DELAY ------  =   202
PERCENT OF VEHICLES INCURRING STOPPED DELAY -----  =    79.5
AVERAGE STOPPED DELAY (SECONDS) -----------------  =     8.3
AVERAGE STOPPED DELAY/AVERAGE TRAVEL TIME -------  =    11.7 PERCENT

DELAY BELOW 10.0 MPH (VEHICLE-SECONDS) ----------  =   9921.0
NUMBER OF VEHICLES INCURRING DELAY BELOW 10.0 MPH  =   236
PERCENT OF VEHICLES INCURRING DELAY BELOW 10.0 MPH =    92.9
AVERAGE DELAY BELOW 10.0 MPH (SECONDS) ----------  =    42.0
AVERAGE DELAY BELOW 10.0 MPH/AVERAGE TRAVEL TIME   =    59.3 PERCENT

VEHICLE-MILES OF TRAVEL -------------------------  =    61.487
AVERAGE VEHICLE-MILES OF TRAVEL -----------------  =      .242
TRAVEL TIME (SECONDS) ---------------------------  =  18017.1
AVERAGE TRAVEL TIME (SECONDS) -------------------  =    70.9
NUMBER OF VEHICLES PROCESSED --------------------  =   254
VOLUME PROCESSED (VEHICLES/HOUR) ----------------  =   1524.0
TIME MEAN SPEED (MPH) = MEAN OF ALL VEHICLE SPEEDS =    14.9
SPACE MEAN SPEED (MPH) = TOT DIST/TOT TRAVEL TIME  =    12.3
AVERAGE DESIRED SPEED (MPH) ---------------------  =    28.3
AVERAGE MAXIMUM ACCELERATION (FT/SEC/SEC) -------  =     4.5
AVERAGE MAXIMUM DECELERATION (FT/SEC/SEC) -------  =     4.3

OVERALL AVERAGE TOTAL DELAY (SECONDS) -----------  =    39.4
OVERALL AVERAGE QUEUE DELAY (SECONDS) -----------  =    29.3
OVERALL AVERAGE STOPPED DELAY (SECONDS) ---------  =     6.6
OVERALL AVERAGE DELAY BELOW 10.0 MPH (SECONDS) --  =    39.1

NUMBER OF VEHICLES ELIMINATED (LANE FULL) -------  =     7

AVERAGE OF LOGIN SPEED/DESIRED SPEED (PERCENT) --  =    87.7
```

2. On two inbound lanes, 45 percent of the vehicles were assumed to be in the median lane and 55 percent of the vehicles were in the curb lane (case 4).

3. In every case, 15 percent of the vehicles turned right, 10 percent of the vehicles turned left, and 75 percent of the vehicles went straight through.

4. On the minor (north-south) approaches, the mean speed was 40 km/h (25 mph) and the 85th percentile speed was 48 km/h (30 mph). On the major (east-west) approaches, the mean speed was 48 km/h and the 85th percentile speed was 56 km/h (35 mph).

5. The arrival headway pattern was described by the negative exponential distribution.

6. Program-supplied values for the percentage of vehicles in each of 10 vehicle classes and the percentage of drivers in each of three driver classes were used (4, p. 33). Vehicle-related values are given in Table 2, and driver-related values are given below:

Driver Class	Driver Type	Driver Characteristic	Perception Reaction Time (s)
1	Aggressive	110	0.5
2	Average	100	1.0
3	Slow	85	1.5

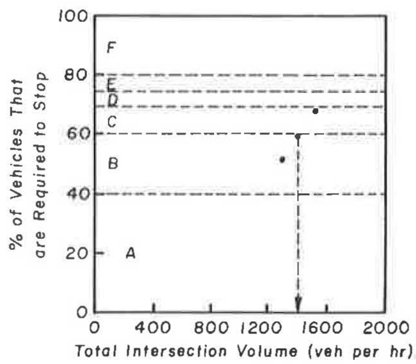


Figure 6. Service volume at level of service B for two-lane by two-lane uncontrolled intersection.

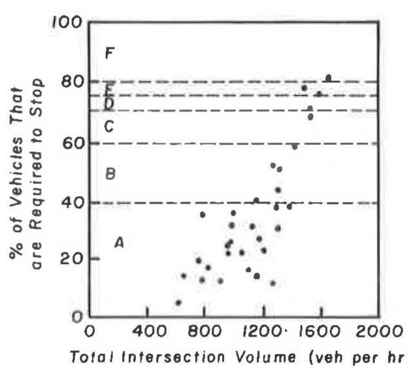


Figure 7. Analysis of two-lane by two-lane uncontrolled intersection.

Table 3. Level of service for total intersection volume of 1600 vehicles/h.

Type of Traffic Control	Lane Configuration: Two-Lane Major, Two-Lane Minor	Two-Lane Major, Four-Lane Minor	Four-Lane Major, Two-Lane Minor	Four-Lane Major Four-Lane Minor
Uncontrolled	D	-	-	-
Yield sign	D or E	-	E	-
Two-way stop	D	B	B	A
All-way stop	E	E	C or D	C

Simulation

Starting with no vehicles in the system, generated driver-vehicle units were positioned at the start of the approach according to the calculated arrival time. Then, depending on the desired speed, destination, traffic condition, and relative position in the intersection area, each unit responded logically to its surroundings. The system was scanned and updated at fixed intervals of 0.5 s. Each unit was processed through the approach. Flow through the system was assumed to attain a steady-state condition in 2 min of real time. Until then, all the movements were simulated but no statistics were gathered. After that, all movements were simulated and statistics were accumulated as each vehicle logged out of the system at the end of the exit. The duration of simulation for this example was 10 min of real time. Figure 5 shows the summary statistics for the intersection used in this example. The intersection was uncontrolled in cases 1, 2, and 3; therefore, the total intersection volume and the percentage of vehicles that were required to stop were used for capacity analysis.

Analysis

Case 1

For a two-lane by two-lane uncontrolled intersection that accommodates a volume of 1600 vehicles/h, Figure 5 shows that the percentage of vehicles required to stop is 79.5 percent. From Table 1, the level of service provided is E.

Case 2

For a two-lane by two-lane uncontrolled intersection that is to operate at a level of service B, the first estimate of volume was 1300 vehicles/h. The percentage delayed in this case, after the model was run in the manner described above, was 51.7 percent. The level of service provided was B, but this is not the maximum volume that can be accommodated. The model was next run with a volume of 1500 vehicles/h. The proportion stopping was now 68.4 percent. The level of service provided in this case was C. Next the model was run with a volume of 1400 vehicles/h. The proportion of vehicles required to stop was now 59.3 percent, which is very close to the upper boundary of level of service B. Thus, it can be stated that the service volume of the two-lane by two-lane uncontrolled intersection operating under a level of service B is 1400 vehicles/h. Figure 6 shows a graph of total intersection volume versus the percentage of vehicles required to stop for these three runs.

Case 3

Analysis on a two-lane by two-lane uncontrolled intersection was conducted by running the TEXAS model with a wide range of volumes. From the summary statistics reported, a graph was drawn for total intersection volume versus the percentage of vehicles that are required to stop (see Figure 7). Horizontal lines that represent levels of service, obtained from Table 1, were superimposed, and a table that relates level of service to total intersection volume was constructed. This table, which is given below, can be used to find the level of service provided at any volume and the maximum volume that can be accommodated at each level of service for a two-lane by two-lane uncontrolled intersection:

Level of Service	Percentage of Vehicles Required to Stop	Intersection Service Volume (vehicles/h)
A	40	1200
B	60	1400
C	70	1500
D	75	1600
E	75	1600

Similar graphs and tables can be constructed for other traffic controls and lane arrangements.

Case 4

To design the intersection so that 1600 vehicles/h can be accommodated while a level of service A is maintained, different lane arrangements and traffic control schemes were tried. The TEXAS model was run with these geometrics and controls with a volume of 1600 vehicles/h until the desired level of service was attained. An efficient and economical way would be to try the most likely arrangement. For a first trial, a two-lane by two-lane stop-sign-controlled intersection was tried. A level of service D was provided, so a four-lane by four-lane, two-way, stop-sign-controlled intersection was tried. The level of service that was now provided was A. Other combinations were tried, but no other lane arrangement and traffic control scheme gave a level of service A.

Table 3 gives a matrix of lane arrangements and control schemes that were run and shows the level of service that would be provided under each scheme. The table can be used in two ways: to determine the level of service of an existing or proposed intersection or to design an intersection to provide any desired level of service. The table is to be used when total intersection volume is 1600 vehicles/h, distributed according to the assumed traffic data given earlier. Similar tables can be constructed for different intersection volumes once the proper distribution is determined.

ACKNOWLEDGMENT

The TEXAS model was developed as part of a research study under the cooperative research program sponsored by the Texas State Department of Highways and Public Transportation and the Federal Highway Administration (FHWA). We wish to thank the sponsors.

The contents of this paper reflect our views, and we are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of FHWA. This report does not constitute a standard, specification, or regulation.

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Integrated System for Urban Traffic Data Collection

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A technique developed at the University of Leeds that combines the collection, analysis, and presentation of data on urban traffic flow in one integrated system is described. Black-and-white aerial photographs of the central area of the city of Leeds were used. The analysis technique involves the use of a coordinate reader and a computer. Data on vehicles in motion and temporarily halted vehicles are collected on a street-by-street basis and fed into the computer on high-speed paper tape. These data are then used to calculate several major

traffic parameters, including directional vehicle spot speed, volume, lane concentration, and headway distribution. The advantages and disadvantages of the technique are presented and discussed. It is concluded that the technique compares favorably with more traditional techniques of urban traffic data collection.

Data analysis has always been the main stumbling block in obtaining traffic data from photography. The very nature of a traffic survey involves the collection of large amounts of data, and this invariably means many hours of tedious work to extract the required information.

Although many photographic surveys have been undertaken in the past, they have dealt mainly with either analysis or data presentation and have not combined the two into a complete technique. Skilled technicians are often required to undertake such surveys, and the information obtained may be rather limited and may not justify the expense involved in its collection.

This paper discusses in detail a technique developed at the University of Leeds that combines the collection, analysis, and presentation of data on urban traffic flow in one integrated system.

AIMS OF THE STUDY

The aims of the work were

1. To use the computing facilities now generally available to reduce analysis time;
2. To develop a complete technique of traffic data analysis that presents the information in an immediately useful form;
3. To obtain results that are sufficiently accurate for traffic engineering purposes, usually ±10 percent (1);
4. To develop a system that can be operated by unskilled or semiskilled personnel; and
5. To obtain information on a wide range of traffic parameters.

It should be noted at this stage that the only vehicles considered were those in motion or at a temporary halt. Information on parked vehicles was not collected.

STUDY AREA

The city center of Leeds was selected as the most convenient site for study. The photography had to be such that it completely covered this area; it was found that only one strip of photographs was necessary for this purpose. The final study area measured approximately 2000 m in an east-west direction and 900 m in a north-south direction. Fifty-eight streets were selected for study.

PHOTOGRAPHIC TECHNIQUE

The size of the area limited the possible techniques to those that involve moving aircraft, and 23×23-cm aerial photographs were used. To accommodate as many photographs as possible of each street that was analyzed in each run of the aircraft, an extended longitudinal overlap was used that was usually in the region of 80 percent. This ensured that each street appeared in at least four consecutive photographs in each flight.

Black-and-white photography was used in preference to color. Preliminary investigations with black-and-white photography showed that individual and particular vehicles could be identified with little difficulty and, of course, black-and-white photography was considerably cheaper.

The flight was carried out during the evening peak period between the hours of 4:00 and 6:00 p.m. on a typical weekday. The area was flown over in an east-west direction and the flights were repeated at approximately 15-min intervals during this evening peak period.

Because of adverse weather conditions, it was not possible to anticipate the actual day on which the photographs were to be taken, and thus it was not possible to arrange a ground survey by which the results could be directly compared with those obtained from the photographic survey.

The specifications for the photography were (a) black-and-white panchromatic paper prints, double weight; (b) a scale of approximately 1:4000; (c) a flying height of 640 m; (d) 152.78-mm focal length of camera; (e) a shutter speed of 0.003 s; and (f) a 4-s time interval between successive photographs. The 4-s time interval was used because it was the shortest possible interval that could be guaranteed by the air survey firm to give the maximum possible overlap when the aircraft was flying as slow as was practicable. The minimum cycling time of the camera prevented shorter intervals of time from being achieved accurately.

ANALYSIS TECHNIQUE

The main objective was to reduce as far as possible the tedious work involved in data extraction while retaining a satisfactory level of accuracy in the results. A D-Mac coordinate reader and the University of Leeds ICL 1906A computer were used in the analysis. The coordinate reader consisted of a table that was capable of reading coordinates to ±0.05 mm; this was interfaced with a high-speed paper tape punch. In addition, a 6X magnifying eyepiece, fitted with crosshairs and attached to the coordinate reader, effectively increased the 1:4000 contact scale to approximately 1:700, which was found to be ideal for identification of vehicles.

Since the intention of the study was to obtain information on several traffic parameters, it was necessary to convert the D-Mac machine coordinates to coordinates of the National Grid Coordinate System so that, although only one section of the total area could be studied at any one time, the results from all the sections could be related to the same coordinate system to give a comprehensive picture of the study area. A linear transformation technique (2) was used in which the photographic coordinates were effectively converted to National Grid coordinates by means of the following equations:

$$X_N = Ax_p + By_p + C \quad (1)$$

$$Y_N = Ay_p - Bx_p + D \quad (2)$$

where

X_N, Y_N = National Grid coordinates,
x_p, y_p = photographic coordinates, and
A, B, C, D = transformation coefficients.

If the National grid coordinates and photographic coordinates of two control points are known, four equations can be formed that enable the transformation coefficients to be evaluated and subsequently used to convert other photographic coordinates to the National Grid. The equations make no allowance for variations in height over the photograph or between control points, and they do not correct for any tilt that may be inherent in the photograph. It was also found that the transformation gave satisfactory results only when the points to be transformed were either on or near the line that joined the two control points.

However, since the study area was the town center of Leeds, where there were only small variations in height, and the photographic tilt was guaranteed to be less than 2°, it was found that, if the distance between control points was not excessive (say, 50 to 100 m), the transformation equations were suitable. The problem of points lying on or near the line that joined the control points was easily solved in this case: The streets that were analyzed were long and narrow and, when the control points were placed on or near the roads, the vehicles were invariably situated on or somewhere near the lines that joined the successive control points.

CHOICE OF CONTROL POINTS

The distance between the control points was an important factor. Previous studies had shown that control points at 50-m intervals gave greater accuracy in speed measurements than control points at 100-m intervals. Control points were therefore set up at approximately 50-m intervals wherever possible (because of the problem of locating suitable control points along the study streets, this value was flexible). The interval was always kept below 100 m. A comprehensive check later showed that, if the distance between control points was less than 100 m, accuracy depended not so much on the distance between points as on the distance each vehicle strayed from the line that joined the two points.

Wherever possible, points were chosen along the side of the study road or, ideally, along the center of the road on traffic islands. Corners or buildings were used, but care had to be taken because buildings tended to mask the control points when they occurred near the edge of the photographs. Corners of curbs were often used because they were easily identified and were at ground level. All the control points chosen were at ground level, and only points that were readily identifiable in the photographs were used.

It is important to note that the control points were marked only on the photographs and on Ordnance Survey plans, not on the ground; their National Grid coordinates were measured from the 1:1250 Ordnance Survey plans, and no fieldwork was required.

A total of about 655 control points were set up. In streets that contained tall buildings on both sides, the control points were set up in pairs—on opposite sides of the road wherever possible—to overcome the masking problem caused by the buildings.

DATA COLLECTION

A street-by-street method of analysis was adopted. All photographs of a street from one strip of photographs were attached to the coordinate reader table, and the coordinates of the control points for that street in the first photograph were taken. The coordinates of the vehicles in that street in the first photograph were then taken lane by lane. Starting with the inside lane of one direction, each vehicle was coordinated in turn by moving the magnifying lens down the lane of vehicles in a direction opposite to the direction of travel. The point on the vehicle that was actually coordinated was the front offside corner. This procedure was repeated for subsequent photographs of the selected street. Each street usually appeared in at least four consecutive photographs.

At the end of the analysis, the paper tape was fed into the computer, and the required information was obtained. The program calculated the National Grid coordinates of each vehicle and used them to calculate several traffic parameters. A special booking form was designed to help monitor the movement of vehicles from one photograph to the next.

PROBLEMS IN THE ANALYSIS

If the same vehicles appeared in each photograph and no vehicles entered or left during the photographic coverage, then the analysis would be straightforward. This situation was, however, uncommon; it was much more usual for vehicles to leave or enter the study street from one photograph to the next. In addition, vehicles do not always enter at one end of the street or leave at the other end. Additions to or subtractions from the flow can occur throughout the length of the street—for example, when a vehicle parks on the street or on streets that are fed by or feed several side roads. Thus, a vehicle that appears in the first photograph of a particular street may not be present in the second but, to avoid confusion, must be retained in the second photograph despite the fact that it is no longer present in the street. Similarly, the first two vehicles in the first photograph may be split by a third vehicle that enters in the second photograph. Here again, to avoid confusion, the presence of the third vehicle must be recorded in the first photograph despite the fact that it has not yet arrived in the street.

To allow for this, the vehicles in each photograph were recorded on a booking form and, at the end of the analysis, the paper tape was input into the computer and a printout was obtained. Editing techniques were then used to insert identifiers at the points in each photograph at which vehicles had entered or left. These identifiers caused the computer program to bypass various sections and move to the next vehicle. This editing ensured that, for example, the seventh vehicle in the second photograph was the same as the seventh vehicle in the fourth photograph.

Editing the files so as to insert vehicles is, unfortunately, necessary. Several attempts were made to introduce a system that would overcome this problem automatically, but none was successful. To obtain traffic information on such parameters as speed, headways, volumes, and densities, it is necessary to keep vehicles in a strict order so that direct comparisons can be made.

At times, particularly in the case of the "rounder" type of vehicle (such as the Volkswagen), the front offside corner of each vehicle was not easily identified. In addition, buses caused problems because of their height, and vehicles were occasionally masked as they passed under bridges. It was not always possible to see the required corner of the vehicle at ground level because of the effect of parallax. In such cases, which were rare, the crosshairs were placed over the point where the corner was estimated to be.

In the photographs taken toward the end of the study period, daylight was beginning to fail and a control point or a dark vehicle was occasionally difficult to identify against a very dark background. This also occurred in areas of heavy shadow, but rarely.

Figure 1. Parameters involved in error analysis.

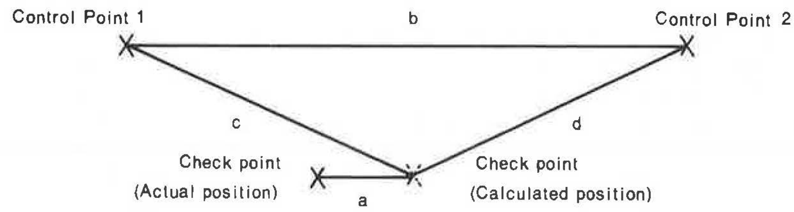


Figure 2. Directional vehicle hours per hour versus directional vehicle kilometers per hour.

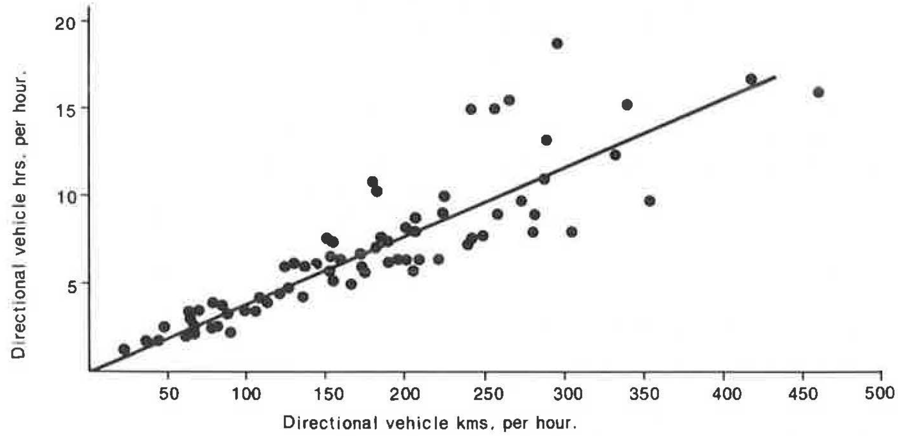
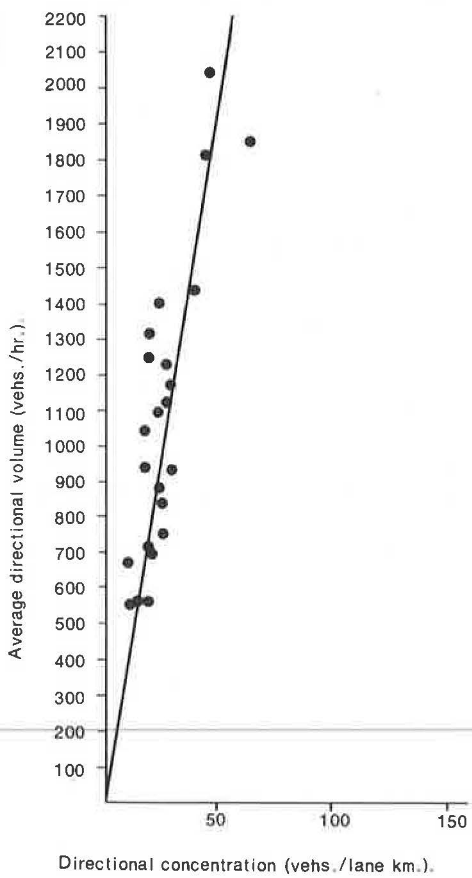


Figure 3. Average directional volume versus directional concentration.



Occasionally, a vehicle from one lane would overtake a vehicle from the same lane. In the strict order of analysis of vehicles, an overtaken vehicle must be analyzed before the vehicle that has overtaken it. Following the principle of lane-by-lane analysis reduced this problem since it was found that the vast majority of drivers remained in the same lane.

The method of analysis used in the program involved the selection of the two control points nearest each individual vehicle. In certain instances, such a method resulted in a vehicle having two control points that were not appropriate. Careful positioning of control points reduces this problem.

Eyestrain became a problem after 3 or 4 h, and the grinding noise emitted by the high-speed paper tape punch also became an annoyance.

A simple three-tier classification that divided traffic into automobiles and light commercial, heavy commercial, and public service vehicles was used on the booking forms. This was found to be useful in later automobile studies and also because the presence of a large, heavy commercial vehicle or public service vehicle greatly helped in the analysis by breaking up the smaller vehicles into manageable groups.

ACCURACY OF THE TRANSFORMATION TECHNIQUE

To test the accuracy of the initial results, a comprehensive check was undertaken on four selected streets. In each of the four streets, several ground-level checkpoints were selected from Ordnance Survey plans. The National Grid coordinates of the points were then measured from these plans by using a 1:1250 metric scale rule. Corners of buildings, walls, and steps were used as well as any distinct features in the road or foothpath surface. Several points that blended in with the background were chosen to represent vehicles that were more difficult to identify because of shadow and lighting problems. These checkpoints were treated as vehicles, and their photographic coordinates were transformed to National Grid coordinates by using the control points on that street. The difference, of course, was that the National Grid coordinates were actually known, and thus the computed values could be compared with the actual values.

Each street was analyzed several times by using different intervals between control points. More than 3200

Figure 4. Directional vehicle kilometers per hour versus mean directional speed (free-flow speed = 59 km/h).

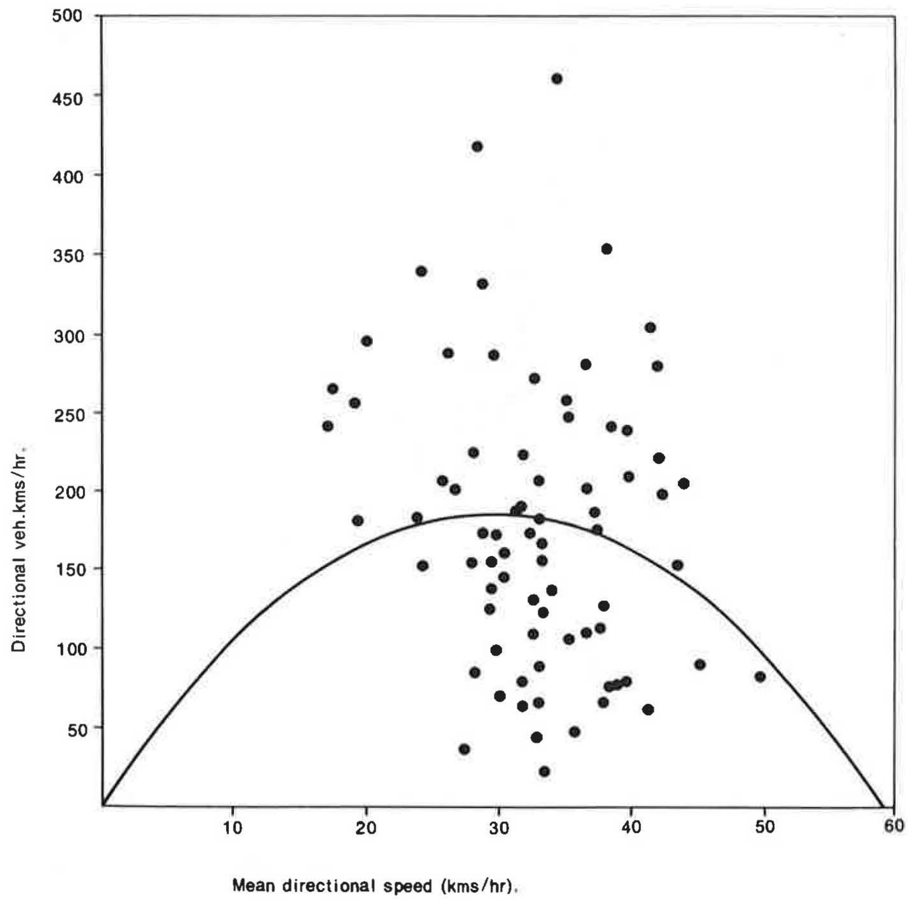
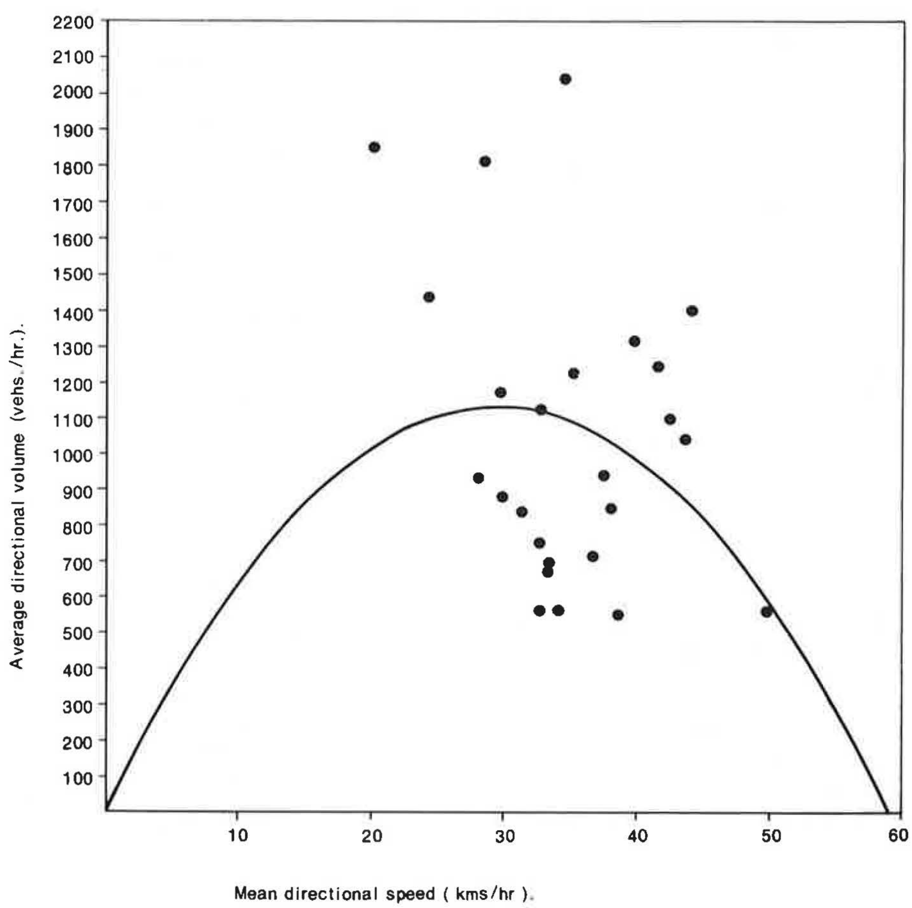


Figure 5. Average directional volume versus mean directional speed (free-flow speed = 59 km/h).



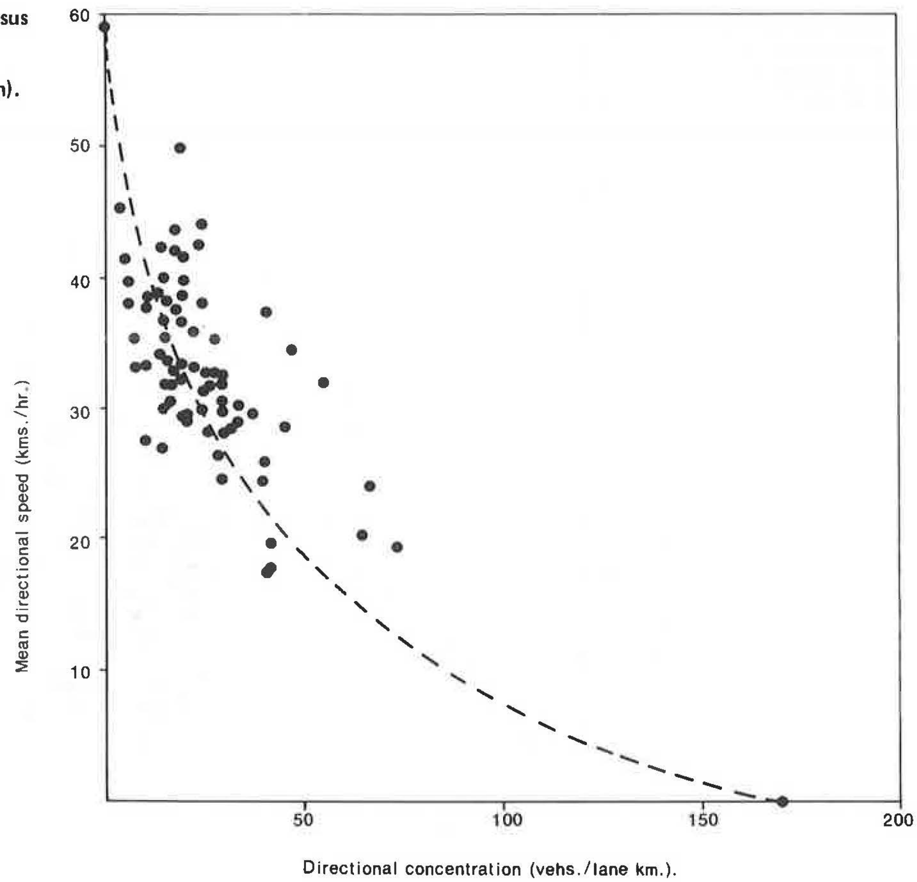


Figure 6. Mean directional speed versus directional concentration (free-flow speed = 59 km/h and saturation concentration = 170 vehicles/lane-km).

Figure 7. Postulated relation for mean directional spot speed versus directional lane concentration.

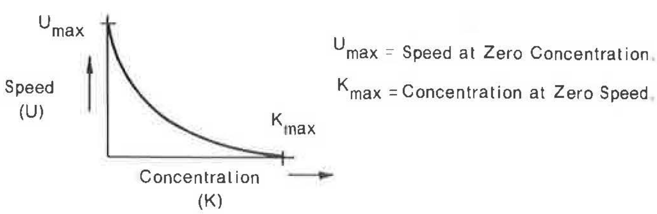
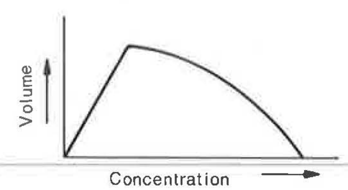


Figure 8. Curve for volume versus concentration.



points were coordinated in this way and analyzed.

Figure 1 shows the parameters involved in the analysis. In the figure,

a = error in the placing of the checkpoint,
b = distance between control points,
c = distance from control point 1 to the checkpoint,
d = distance from control point 2 to the checkpoint, and
e = (c + d) - b.

Values of a, b, c, d, and e were obtained for each checkpoint.

A study of the results showed that the e-values of Figure 1 were the critical factors in the resulting error values. The e-value is the difference between the distance between control points and the sum of the distances from the control points to the checkpoint. Three ranges of e-values were considered: less than 15 m, between 15 and 30 m, and greater than 30 m.

The effect of the e-values and the positioning errors of the checkpoints (distance a in Figure 1) were considered with respect to determination of vehicle spot speed. Spot speed was calculated from the distance traveled from one photograph to the next in the 4-s time interval.

The table below gives the standard errors in vehicle position and spot speed that occurred at different values of e (1 m = 3.3 ft; 1 km = 0.62 mile):

e (m)	Standard Error in Position (m)	Standard Error in Spot Speed (km/h)
< 15	±0.96	±1.22
15-30	±1.26	±1.60
> 30	±2.09	±2.66

In practice, any one error will have a 68 percent chance of being less than the figures given in the table. These figures apply to distances of no more than 100 m between control points.

The speed values give the best indication of the accuracy of the technique. If the e-values can be kept below 30 m or, better still, below 15 m, the errors that occur in the speed values will be less than those that normally occur on the speedometers of vehicles. The e-value and the limitation it sets provide a very useful guide in setting up control for future surveys; in this study, it provided the basis for deciding on the positions of the control points.

TRAFFIC PARAMETERS

The computer program used during the study produced the following traffic parameters from the data:

1. The National Grid coordinates of each vehicle,
2. The distance traveled by each vehicle from one photograph to the next,
3. The cumulative directional distances traveled by all the vehicles in each street for each photographic flight (this was expressed as directional vehicle kilometers per hour),
4. The spot speed of each vehicle from one photograph to the next,
5. Mean directional spot speeds of all vehicles in each street for each flight,
6. Vehicle trajectory curves,
7. The directional lane headways of the vehicles in each street for each flight,
8. The mean directional volume of vehicles in each street for each flight,
9. The directional lane concentration of vehicles in each street for each flight,
10. The cumulative directional time spent by all vehicles in each street for each flight (expressed as directional vehicle hours per hour), and
11. Traffic-density contours for the whole study area for each flight.

DISCUSSION OF RESULTS

Figures 2-6 show the graphs that were drawn from the traffic parameters obtained. Only the free-flowing cases were considered—that is, those cases in which traffic was moving easily and without restriction or interference from other road users. The three road types considered were dual carriageways, normal two-way streets, and one-way

Figure 9. Density contour map for Leeds central area.

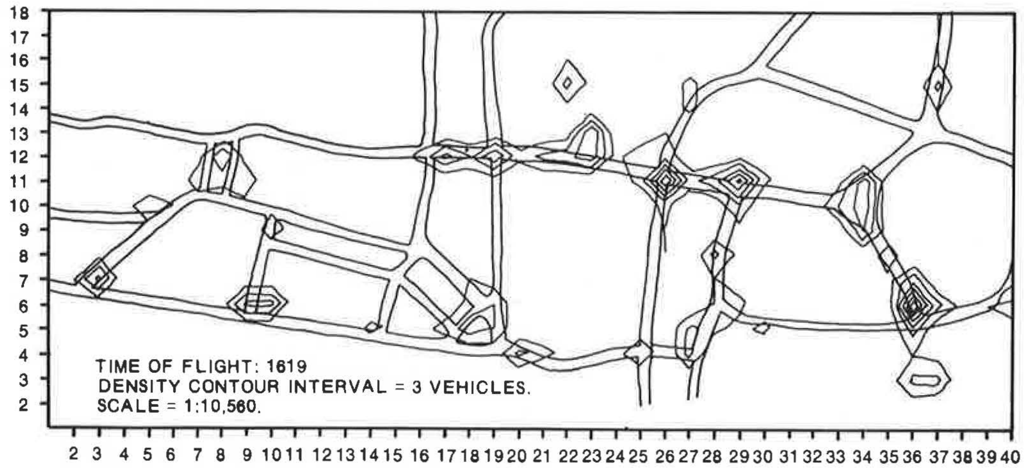
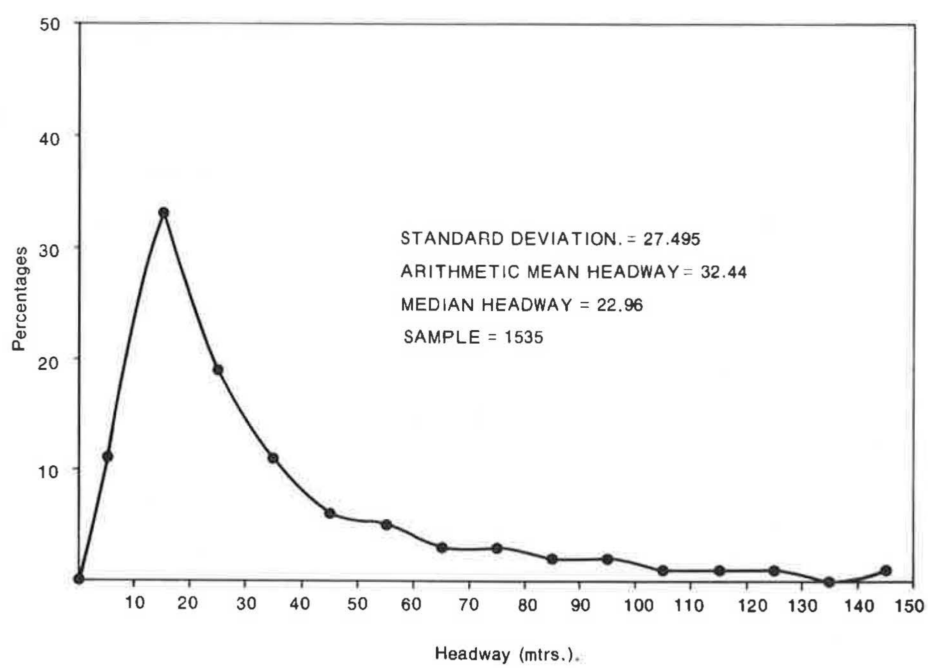


Figure 10. Typical headway distribution.



streets. Figures 2-6 present data for dual carriageways.

Good linear relations were obtained when directional vehicle hours per hour was plotted versus directional vehicle kilometers per hour (Figure 2) and when average directional volume was plotted versus directional lane concentration (Figure 3). Parabolic curves were fitted to the results obtained by plotting directional vehicle kilometers per hour versus mean directional spot speed (Figure 4) and average directional volume versus mean directional spot speed (Figure 5), but large scatters were obtained.

Figure 6 shows the nonlinear relation obtained when mean directional spot speed was plotted versus directional lane concentration (the broken curve is only a trend line and not the true curve fitting the points). This relation was found to be similar to one postulated by Drake and others (3) and May and Keller (4), which is shown in Figure 7. This gave a possible reason for the scatter obtained for the parabolic curves since, for such speed-concentration curves, the volume-concentration curves are of the form shown in Figure 8—that is, a straight-line section running into a parabolic curve as the concentration increases.

The volume-concentration results obtained in this study were linear in form, and this would suggest that they represent part of the straight-line section shown in Figure 8. This is substantiated when it is remembered that this linear relation was obtained for free-flow conditions (conditions that have low concentration values). In addition, the speed-volume curve associated with the curve shown in Figure 7 tends to differ from the traditional parabolic form and consists of a combination of a straight line and a parabola. The curves obtained for speed versus volume and speed versus vehicle kilometers per hour in this study did show a large scatter when a parabolic curve was fitted, which may result from the straight line-parabola effect.

In addition to the graphs, good results were obtained for the traffic and automobile-unit density contour maps. Figure 9 shows one of the density contour maps based on numbers of vehicles only, but similar graphs can be produced that allow for any specified system of automobile units. Headways were easily calculated from the data; Figure 10 shows a typical headway distribution for dual carriageways. Successful vehicle trajectory curves were also produced.

Despite the fact that no ground survey was undertaken as a check, information on many traffic parameters was obtained that shows good correlation with some established theories of traffic flow.

DISADVANTAGES OF THE TECHNIQUE

The problem of data analysis still remains to some extent even though computers and coordinate readers greatly reduce the time involved in performing calculations on the data. The data cannot be fed directly from the coordinate reader output into the computer, however, and certain amendments are necessary if a strict vehicle order is to be maintained.

The maintenance of this vehicle order is very important because, to compare individual vehicles from one photograph to the next, it is vital that the whereabouts of each vehicle in relation to all the other vehicles is known. Of course, this requires an operator who is skilled in computer-program editing methods. This part of the process is comparable to the data-checking process undertaken in manual traffic surveys. The time involved is not excessive and, once the data are corrected, they are ready for computer analysis.

The small time coverage of traffic movement on individual streets hampered studies of traffic volume. It is difficult, however, to reduce the time interval by using conventional aerial survey cameras. Thus, despite the correlation of the volumetric curves to theoretical hypotheses, the photographic technique is not easily adaptable to the measurement of volume. It seems unreasonable to calculate values from such small samples unless the time coverage can be increased.

It is unfortunate that no ground survey could be undertaken to act as a check on the photographic technique. Such a survey would have been very useful and might have thrown some light on the problems encountered during the determination of volumes.

Specialized equipment is required in the analysis. Coordinate readers and computing time are expensive, and the ideal situation would be to have a direct link to the main computer from which the editing of the data could be undertaken.

ADVANTAGES OF THE TECHNIQUE

The accuracy of the method has been shown to be more than adequate for traffic studies. More careful positioning of the control points could increase the accuracy even further if it were necessary. A very important aspect of the study was that all the control points were established from the aerial photographs used in conjunction with the 1:1250 Ordnance Survey plans without fieldwork. The National Grid coordinates of the control points were read directly from the plans, and the accuracy with which these coordinates could be read was clearly inferior to the accuracy that could be obtained by performing an accurate field survey to establish the control. Despite this, sufficient accuracy was obtained.

For a smaller area—one that involves perhaps 30 vehicle-carrying streets—it is probable that the whole study could be completed by one person less than one month after the photographs were received. This must be compared with the number of people and the time required to complete alternative ground surveys.

One great advantage of a photographic study is the amount of supplementary information that can be gleaned from the photographs. This study has been confined to moving traffic; no observations have been made of parked vehicles, which make an ideal subject for study by means of this type of photography. The photography also contains a wealth of other information that may be useful to those outside the field of traffic engineering.

Many other problems associated with moving vehicles could be investigated by using the same blocks of photography, and other highway problems—such as road widths, road markings, location of possible new roads, use of derelict land for parking, surveys of street furniture, and areas of heavy congestion shown by the traffic density contour maps—can be studied from the photography.

The short sampling time can be an advantage. Although, during free-flow conditions, a sampling time of less than 1 min could yield so few vehicles as to be statistically unreliable (as, for example, in the volume calculations undertaken in this study), the use of such a short increment of time ensures that an almost instantaneous picture of flow conditions is provided. Vehicles in the sample can only be a few seconds apart in time, and this results in a limitation on the possible changes in flow characteristics that might take place during longer sampling periods.

SUMMARY

This study has shown the technique used to compare favorably in many ways with more traditional techniques. Problems of data extraction have been eased if not totally eliminated.

A comprehensive check has shown the level of accuracy to be suitable for traffic studies, and an analysis of the results has shown that in many cases they agree with established theories. Deviations from these theories have occurred only where the particular theory itself is in doubt.

Several types of surveys—e.g., origin-destination studies—would, however, be difficult to undertake, and volume studies present problems. The photographs provide a permanent record of traffic conditions at that time and can be referred to later as and if required. The technique can be undertaken by unskilled or semiskilled personnel, and an accurate analysis is quickly obtained.

If some method could be found to eliminate the necessity of maintaining a strict vehicle order and thereby allow the data obtained from a coordinate reader to be read directly into the computer without any need for editing, the method could have many applications and could effectively replace some traditional ground-survey techniques. Even if one allows for this drawback, however, the photographic technique is a feasible one and represents a definite alternative to conventional ground methods.

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Measurement of the Performance of Signalized Intersections

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A survey technique for the measurement of the performance of signalized intersections is described. The technique is simple to perform in the field and produces the following types of information: (a) vehicle delay and its variability, (b) pedestrian delay and its variability, (c) vehicle flow rate, (d) total number of effective vehicle stops, and (e) a complete record of signal phasing and timing. Details are given for implementation of the technique as a conventional field survey, an "instant analysis" field survey, or a simulation model subprogram. The use of the survey technique in the evaluation of schemes for bus priority signalization is also discussed.

Although there has long been an interest in the performance level of intersections, especially signalized intersections, this interest has been heightened in recent years by the necessity to ensure that the existing transportation system operates at peak efficiency. This concentration on the performance of existing systems has been labeled transportation system management (TSM) and covers a wide variety of techniques. One of the major TSM techniques is managment of the system so as to give priority to high-occupancy vehicles. Examples of such priority techniques are priority lanes and bus priority signal systems. Such priority systems give preferential treatment to high-occupancy buses, usually at the expense of low-occupancy automobiles. However, before such a scheme is implemented or continued beyond a demonstration period, an assessment should be made of the relative impacts on various types of vehicles. In the case of bus priority signals, this assessment involves a survey of intersection operating conditions and performance levels.

The measurement of the level of performance of an intersection has been an area of concern in traffic planning almost since the birth of the profession. One can trace the attempts of traffic engineers to grapple with this problem through the works of several authors (1-6). In this evolution of techniques of performance measurement there have been two major variables: (a) the definition of criteria for level of performance and (b) the physical technique for obtaining such a measurement.

Indirect measurements of performance level that have been used include load factor (7), intersection flow ratio (8), and degree of saturation (9). Direct measurements of performance level include vehicle delay (however defined) and proportion of vehicles stopped. Reilly and others (6) describe the various definitions of delay and conclude that the most appropriate definition to use is that of approach delay, which includes not only the delay incurred by a vehicle while actually stopped but also the delay incurred while the vehicle is decelerating and accelerating as a result of the intersection operation. Their definition of the proportion of vehicles stopped is, as the name implies, the number of vehicles that come to a complete halt (no matter how many times) divided by the total number of vehicles crossing the stopline. They also classify the survey techniques into four types: point sample, path tracing, input-output, and modeling. They conclude that the point-sample method is the most desirable.

This paper addresses the question of devising a survey method that is particularly suited to the measurement of appropriate performance levels at a bus-priority-signalized intersection.

EVALUATION OF BUS PRIORITY SYSTEMS

The implementation of bus priority systems may be seen as an attempt to achieve either or both of the following objectives:

1. To improve the person-carrying capacity of a section of roadway and
2. To improve the level of service offered by buses in comparison with automobiles so as to induce a change in mode use along the route.

Either of these objectives may result in a reduced level of service for private automobiles in order to improve the level of service of buses.

The purpose of an evaluation study is to ensure that the degradation in the level of service offered to automobiles does not exceed the improvement in the level of service offered to buses (at least not beyond an acceptable level determined by policy makers). On what basis, then, should this evaluation be performed?

A previous study (10) has proposed an evaluation framework within which bus priority systems may be evaluated. This evaluation framework, shown in Figure 1, considers various evaluation methodologies as lying within a three-dimensional space. This space, with dimensions termed breadth, width, and depth, defines the complexity and completeness of the evaluation procedure. Breadth refers to the number of groups in the community included in the analysis. For bus priority systems, appropriate groups might include bus passengers, automobile drivers and passengers, pedestrians, and nonusers of the facility. Width refers to the geographic area over which the evaluation extends and, for bus priority studies, might consist of an intersection, a link, a route, or a network. Depth refers to the number of factors considered in the evaluation. Factors suggested as appropriate for bus priority studies include travel time delay, travel time variability, operating costs, energy consumption, pollution emissions, safety, mode choice, and distributional impacts.

Obviously, the extent of the evaluation will vary from location to location. Its degree of sophistication should be commensurate with the anticipated magnitude of the costs and benefits of the priority system. But it will be shown that, by using the survey technique described in this paper, a reasonably sophisticated analysis can be performed without vast expenditures on data collection or analysis. Specifically, this paper describes a technique that can be used to obtain data for the evaluation of an intersection signalized for bus priority. This evaluation will be broad enough to include bus passengers, automobile passengers, pedestrians, and nonusers; wide enough to encompass all approaches to a single intersection, from which the results may be used as input to route and network studies; and deep enough to include travel time delay, travel time variability, operating costs, energy consumption, and pollution emissions.

The survey method yields five distinct characteristics on each approach: average delay per vehicle, standard deviation of delay per vehicle, average number of stops per vehicle (as distinct from the proportion of vehicles that stop), flow rate, and sequence and timing of traffic signal indications on each approach. The survey technique uses, in the words of Reilly and others (6), a point-sample method. The time between sampling points, however, is not constant but is synchronized with the changing of signal aspects on each approach. This has the advantage of enabling the signal phasing to be obtained in the same survey while at the same time reducing the workload on survey personnel, who then normally have to record queue length only once per cycle (i.e., about once every 90 s) instead of once every 13 or 15 s as required by the method of Reilly and others. As will be seen later, the survey is also simpler in that only stationary queues are counted and not queues that are being shortened at the front by vehicles that move off after a green signal is shown.

SURVEY THEORY

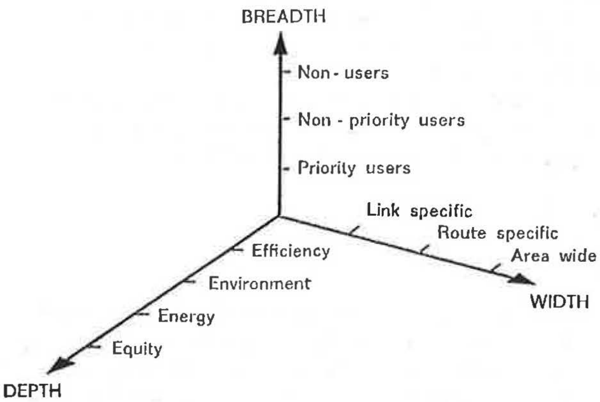
The starting point for the development of the survey theory is the idealized concept of intersection behavior used by May (11) and Sagi and Campbell (4). Unlike May, however, who considered arrival and departure rates to be constant over time, this survey method allows arrival and departure rates to vary from cycle to cycle and only assumes that, within each cycle, arrival and departure rates are linear. The assumption of linear arrival rates within each cycle may be questioned when significant upstream bunching occurs (4, 5). However, as long as the bunch arrivals are not synchronized with the phasing of the traffic signals, this bunching effect should even out over many cycles. One would not expect synchronized bunches along a route of isolated intersections that have vehicle-actuated signalization (this survey method was originally designed for such intersections).

Consider, then, the passage of a vehicle through a signalized intersection, as shown in Figure 2. Vehicle A arrives during the green period and proceeds through undelayed. Vehicle B arrives during the red period, decelerates, and comes to a complete stop. When the signal turns green, vehicle B accelerates to cruising speed and leaves the intersection area. Vehicle C arrives during the red (or green) phase and, finding a vehicle stopped ahead, slows down in preparation for a complete halt. However, the vehicle in front moves off before vehicle C reaches it, and so vehicle C accelerates back to cruising speed without coming to a complete halt.

In calculating delay, it has been shown (12) that, by considering vehicles with infinite acceleration and deceleration rates (i.e., squaring off the trajectory diagrams), the approach delay is equal to the length of the horizontal sections of the trajectories, as shown in Figure 3. This delay includes both the time that a vehicle is stopped, if at all, and the time lost in acceleration and deceleration maneuvers.

Consider now the trajectory diagram associated with one cycle of a set of traffic signals where the input and output flow rates, in that cycle, are constant. Figure 4 shows how the flow in this cycle can be represented by a family of trajectory lines made up of inclined and horizontal sections. The delay to each vehicle is represented by the horizontal section of each trajectory, and the total delay to all vehicles is the summation of the lengths of the horizontal lines. If one considers the flow to be continuous rather than discrete, the total delay can be represented by the area of the triangle that envelops the horizontal lines.

Figure 1. Framework of evaluation for bus priority systems.



To calculate this area, one must specify the location of the triangle apexes in time and space. Points A and B are easily specified. Both are located at the stopline or the front of the queue. Point A denotes the start of the red period, and point B denotes the start of the effective green time. The location of point C is a little less definite. Conceptually, it represents the time and queue position at which arriving vehicles are no longer influenced by the previous red phase. In practice, however, it is not simple to determine whether or not an arriving vehicle was delayed by the previous red phase. One must determine not simply whether the vehicle stopped but whether the vehicle slowed down at all because of the previous red phase. This calls for a degree of judgment that is not usually found in relatively inexperienced observers. What is needed is a rela-

Figure 2. Typical trajectory diagrams for the passage of three vehicles through a signalized intersection.

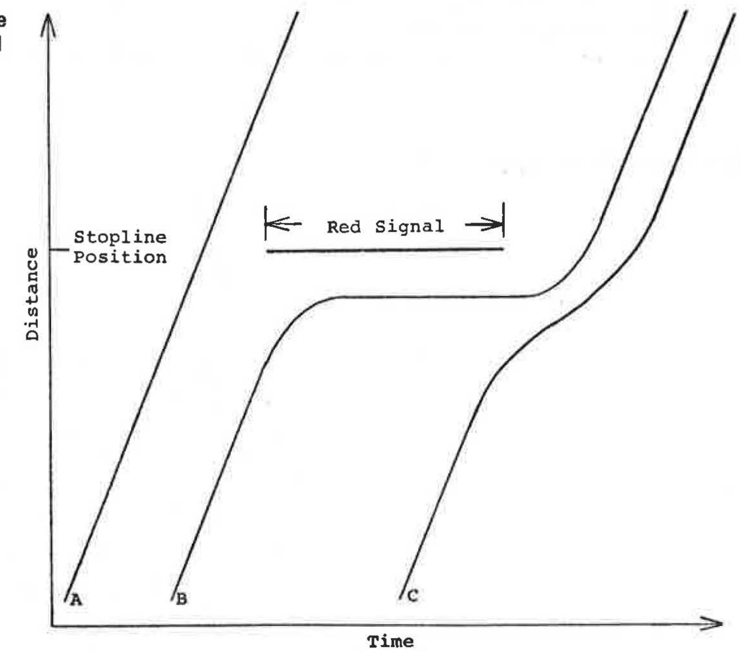
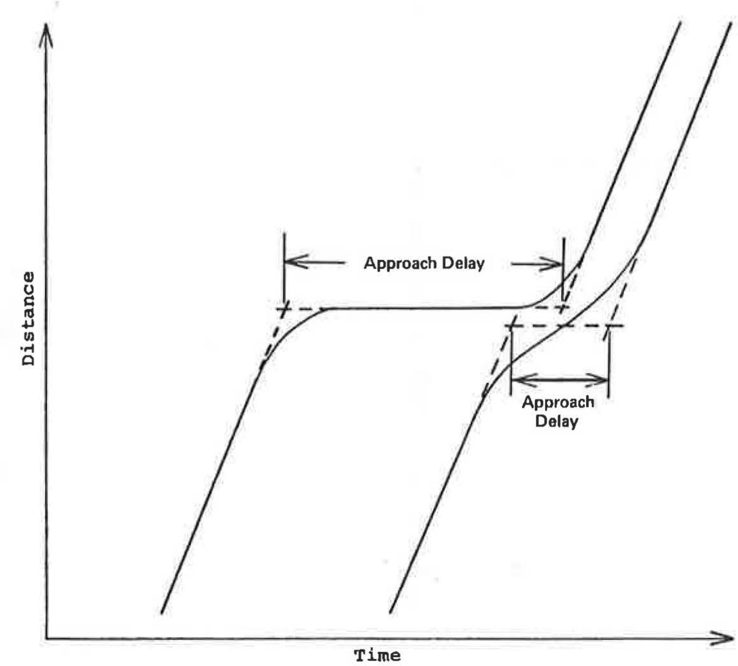


Figure 3. Approach delay calculated from idealized trajectory diagrams.



tively simple measurement that can be related to the position of point C.

One measurement that is relatively easy to obtain is the length of the queue at the time when the green phase begins (shown by BE in Figure 4). An associated measurement is the time at which a vehicle at the end of this queue crosses the stopline after it moves off in the green period (assuming an undersaturated cycle), as shown by point D in Figure 4. These values are sufficient to perform this survey and are summarized in Figure 5. The four values recorded are the time at the start of the red period, the time at the start of the green period, the queue length at the start of the green period, and the time at which a vehicle at the end of this queue crosses the stopline.

From these measurements, the position of C may be estimated graphically as follows (Figure 5). Join points A and E and continue the projection beyond E. From E, construct a horizontal line that represents this stopped vehicle. To represent a vehicle moving at cruise speed, construct an inclined line from D to intersect the horizontal line through E at F. Point F represents the time at which the last vehicle in the queue at time B starts to move off. From B, draw a line through F to intersect AE at C. Point C is defined as before and represents the third corner of the delay triangle.

The vertical distance from point C to the stopline (i.e., the length of queue affected by the red signal) can be expressed mathematically as follows:

$$Q_T = Q_G\,[1/(1 - a \times m)] \qquad (1)$$

where

Q_T = total queue length affected by the red signal,
Q_G = queue length at the start of the green period,
a = average arrival rate at the end of the queue, and
m = average move-off time at the head of the queue.

$$a = Q_G/AB \qquad (2)$$

$$m = (BD - kQ_G)/Q_G \qquad (3)$$

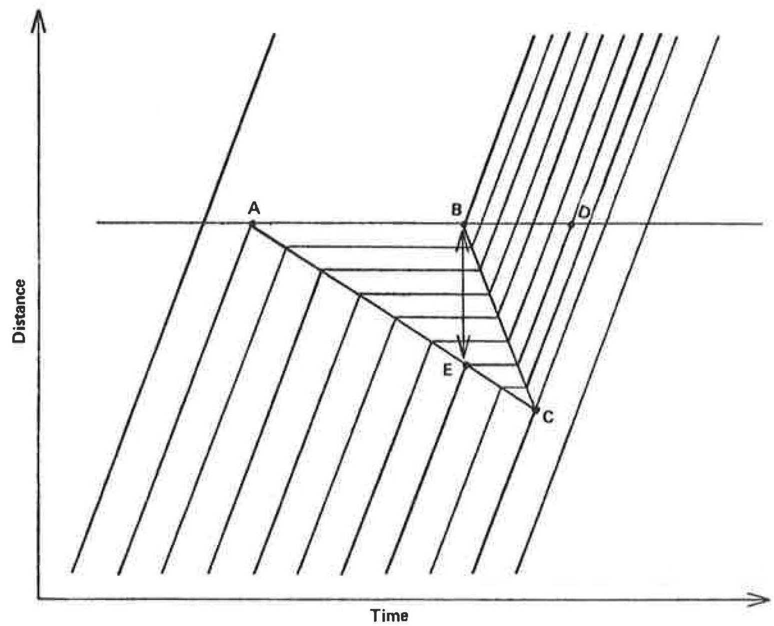


Figure 4. Trajectory diagram for one cycle of traffic signals.

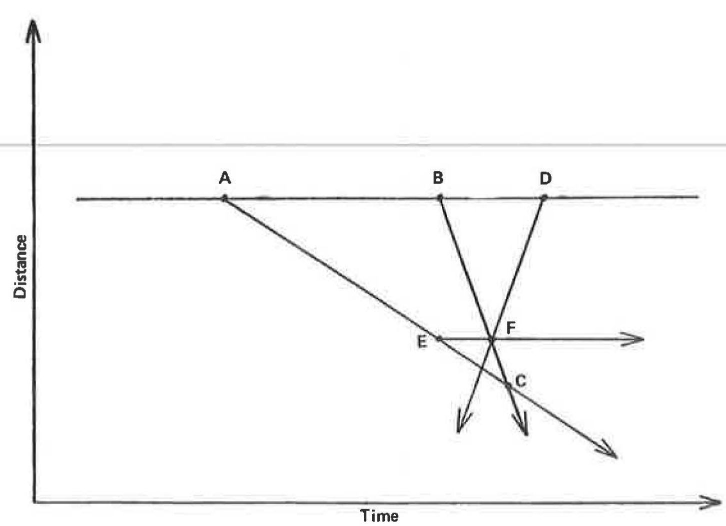


Figure 5. Summary of survey measurements.

where AB, BD = times as shown in Figure 5 and k = time to travel one vehicle spacing at cruise speed. Thus,

$$Q_T = Q_G \{ 1/[1 - (BD - kQ_G)/AB] \} \quad (4)$$

Given this estimate of Q_T in terms of the values measured in the survey, it can be seen from Figure 5 that the delay experienced by each vehicle varies from AB (the effective red period) for the first vehicle in the queue to zero for the Q_Tth vehicle in the queue. This can be redrawn to show vehicle delay as a function of vehicle arrival order (see Figure 6). The total delay for this cycle is then given as the area of this triangle:

$$\Sigma d = (1/2) \times AB \times Q_T \quad (5)$$

where d = individual vehicle delay.

An important indicator of level of performance for a bus priority intersection is the variability in delay experienced by various vehicle streams. It is possible, by using the data collected in this survey of queue length, to calculate the variance in travel delay.

Consider the delay diagram shown in Figure 6. The variance in delay within each cycle may be calculated by

$$\sigma^2 = (\Sigma d^2/n) - (\Sigma d/n)^2 \quad (6)$$

where d = individual delay and n = vehicle flow ($n \approx n - 1$).

It has already been shown that Σd is the area under the curve in Figure 6. Similarly, Σd^2 is the area under the curve in Figure 7, which shows d^2 as a function of vehicle arrival order. Thus,

$$\Sigma d^2 = (1/3) \times (AB)^2 \times Q_T \quad (7)$$

The flow within each cycle may be determined as a function of the measured variables by

$$n = (Q_G \times AD)/(AB - kQ_G) \quad (8)$$

This equation expands the number of vehicles that arrive at the end of the queue during the red period to give the number of vehicles that cross the stopline during the cycle.

To obtain statistics over a number of cycles, one simply sums, over all cycles, the values of Σd, Σd^2, and n obtained from each cycle and, at the end of the desired period, calculates values as follows:

Total delay = $\Sigma(\Sigma d)$.
Total flow = Σn.
Average delay = $\Sigma(\Sigma d)/\Sigma n$.
Variance in delay = $[\Sigma(\Sigma d^2)/\Sigma n] - [\Sigma(\Sigma d)/\Sigma n]^2$.

Another statistic of particular interest, especially in considerations of energy or pollution, is the total number of vehicle stops. Intuitively, one might consider that the total number of vehicle stops is simply ΣQ_T. However, although in our model the assumption of infinite acceleration and deceleration rates makes no difference to the calculation of delay, such an assumption does affect calculation of the actual number of stops. It must be remembered that, in real life, not every vehicle that is delayed is brought to a complete halt (as shown by vehicle C in Figure 2). Thus, a correction must be made to ΣQ_T to account for those vehicles that do not come to a complete halt.

It is easily shown that, with a cruising speed of V m/s and an acceleration-deceleration rate of a m/s^2, vehicles that have delays less than V/a seconds do not experience complete halts. In fact, the change in speed is linear between zero when delay is zero and V when delay is V/a.

In calculating energy consumption and pollution emissions it has been shown (13,14) that they are related to intersection conditions by a common formula,

$$E = \alpha D + \beta s \quad (9)$$

where

E = energy consumed (or emissions),
D = delay,
S = number of stops, and
α, β = conversion coefficients.

Number of stops S is considered to be complete stops from an initial speed. Figures 8 (15) and 9 (14) show the effect of initial speed on the energy consumed and the pollutants emitted. In each case, it can be seen that the relation is approximately linear. Thus, the energy consumed or the pollutants emitted are roughly proportional to the change in vehicle speed irrespective of the initial speed. Thus, vehicles that have delays less than V/a seconds will have energy consumption and pollution characteristics that correspond to their changes in speed. These vehicles can thus be considered fractions of a complete stop with respect to energy and pollution.

The effect of these incomplete stops is to reduce the calculated number of stops, as given by Q_T, according to the number of vehicle stops with delays less than V/a. With a triangular delay diagram (Figure 6), the reduction in the number of stops will be determined solely as a function of the maximum delay. The reduction coefficients do not depend on the number of vehicles delayed but only on the maximum delay in each cycle.

It can be shown that, provided the maximum delay per cycle is greater than V/a ($\approx$10 s), the equation for the stop-number reduction coefficient C is

$$C = [AB - (V/2a)]/AB \quad (10)$$

where

AB = maximum delay in cycle,
V = cruising velocity, and
a = acceleration-deceleration rate.

For typical conditions, the reduction coefficient will be in the range 0.85 to 0.95. Thus, the final statistic of interest may be expressed as

$$\text{Number of stops} = \Sigma\Big(\{Q_T \times [AB - (V/2a)]\}/AB\Big) \quad (11)$$

So far, the analysis has assumed that all vehicles that form a queue are cleared across the stopline before the next red phase starts. But this is not the situation in heavy flow conditions, where vehicles may be forced to stop at least twice before clearing the lights. For the case of oversaturated cycles, a more general form of Figure 6 is shown in Figure 10, and a more general form of Figure 7 is shown in Figure 11.

By use of arguments similar to those used in the undersaturated situation, the following general equations may be derived:

Figure 6. Delay versus vehicle arrival order.

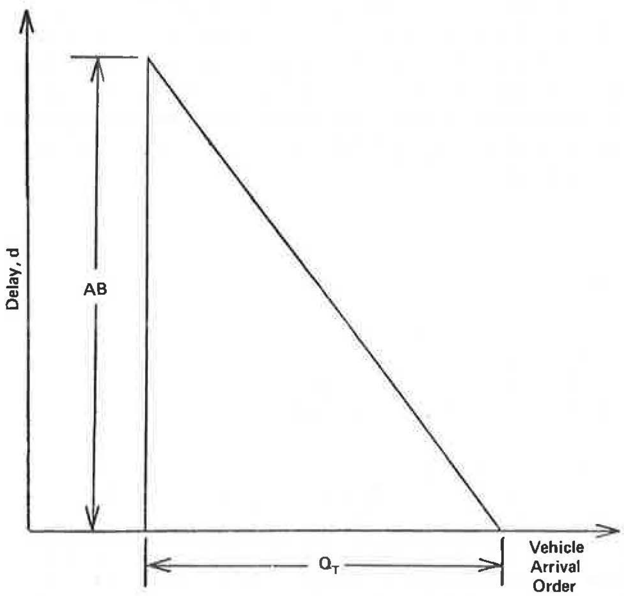


Figure 7. Square of delay versus vehicle arrival order.

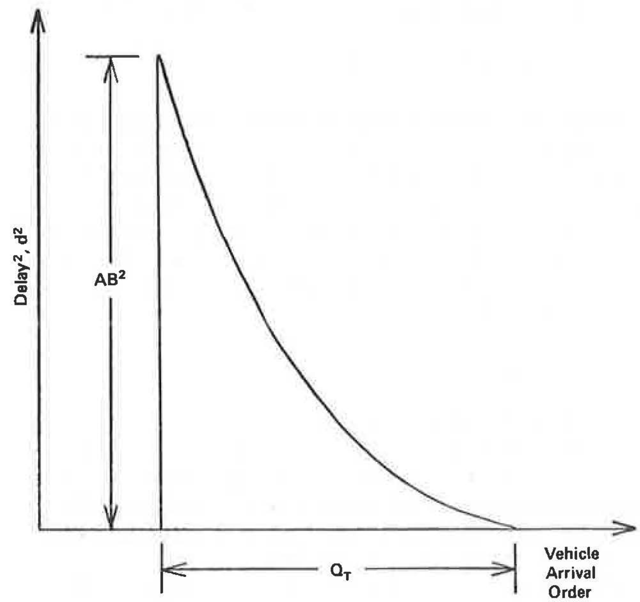


Figure 8. Fuel consumed by vehicles coming to a complete halt from various initial speeds.

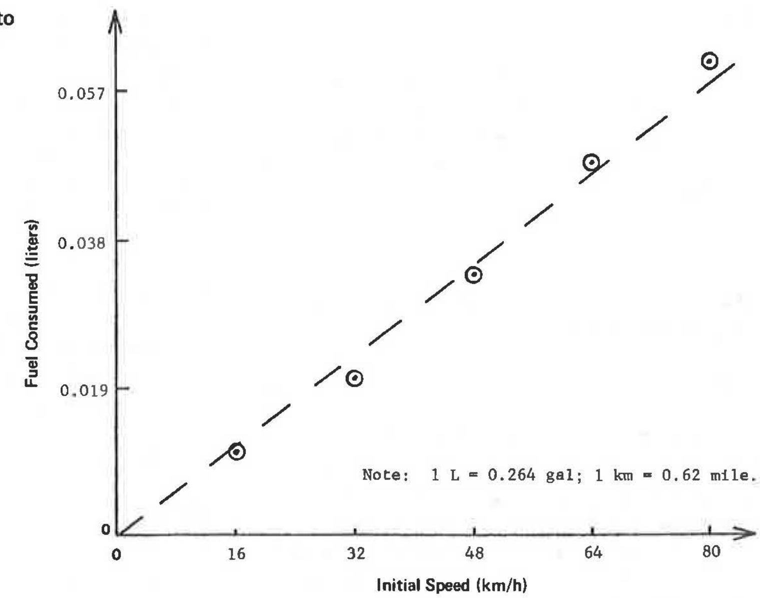


Figure 9. CO emitted by vehicles coming to a complete halt from various initial speeds.

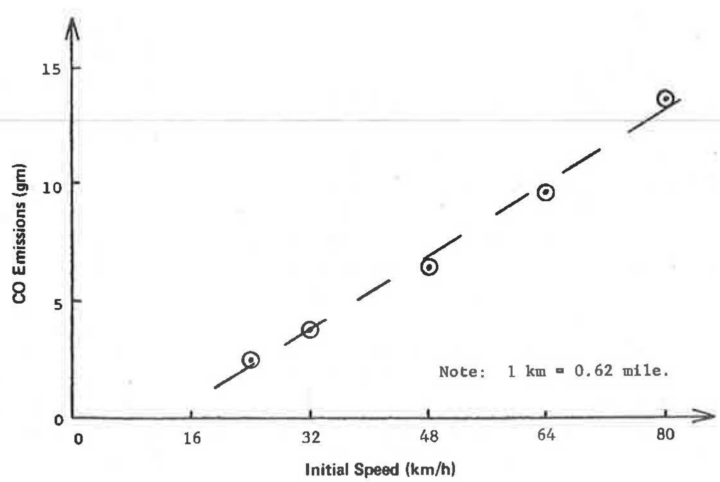


Figure 10. General diagram of delay versus vehicle arrival order.

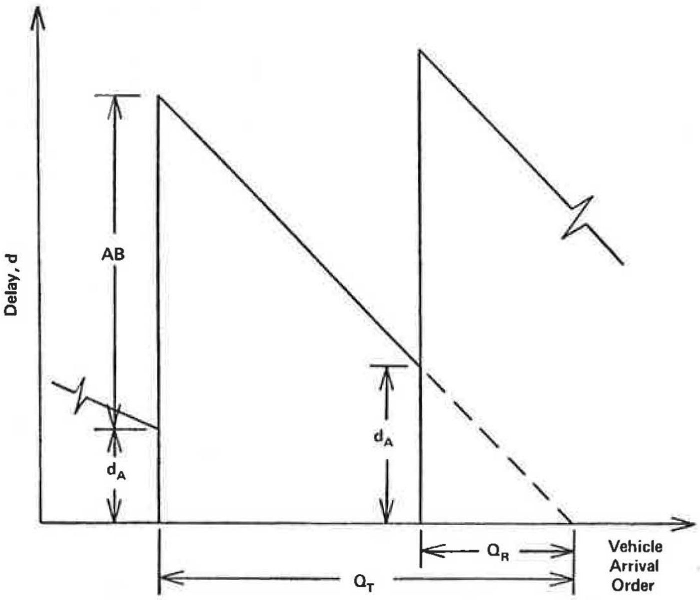
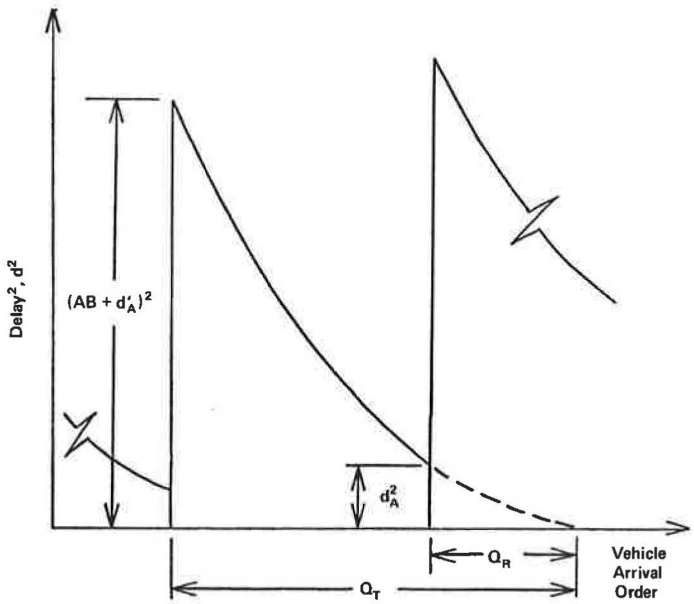


Figure 11. General diagram of square of delay versus vehicle arrival order.



$$Q_T = Q_G \left[1/\left(1 - \{ [(Q_G - Q_R')(BD - k(Q_G - Q_F)]/AB \times Q_G \}\right) \right] \quad (12)$$

where Q_F = number in the queue ahead of, and including, the last vehicle in the queue at the start of the green when a new queue forms at the end of the green and Q_R' = total number in the queue held over from the previous cycle.

It should be realized that in an oversaturated cycle the last vehicle in the queue at the start of the green may not reach the stopline before the light turns red. In this case, the recording technique is modified so that, instead of recording the time at which this vehicle crosses the stopline, it records the number of vehicles in front of, and including, this vehicle that are stopped by the red light. So, of the two variables defined above, Q_F is recorded and Q_R' is calculated from measured variables as

$$Q_R = \begin{cases} Q_T - (1/k)[D - (Q_T - Q_R')/(Q_G - Q_R') \times AB] & \text{if } Q_R > 0 \\ 0 & \text{if } Q_R < 0 \end{cases} \quad (13)$$

where Q_R = total number in the queue held over until the next cycle and D = time at which the signal turned red at the end of the green. In the first oversaturated cycle, Q_R' will be zero, and thus the recursive nature of the above equation is broken.

It can be seen in Figure 10 that the maximum delay to the first vehicle in the queue is made up of two components: (a) the delay caused by the red period of the present cycle and (b) the delay carried over from the previous cycle (d_A'). The additional delay d_A that is carried over to the next cycle is given by

$$d_A = (Q_R/Q_T)(AB + d_A') \quad (14)$$

Again, the recursive nature of this equation is broken in the first oversaturated cycle.

The total delay in this cycle is given by

$$\Sigma d = (1/2)(AB + d_A' + d_A)(Q_T - Q_R) \quad (15)$$

It can be seen in Figure 11 that

$$\Sigma d^2 = (1/3)[(AB + d_A')^2 - d_A^2](Q_T - Q_R) + d_A^2 (Q_T - Q_R)$$
$$= (1/3)[(AB + d_A')^2 + 2(d_A^2)](Q_T - Q_R) \quad (16)$$

Referring back to Figure 10, the flow in each cycle can be expressed as

$$n = \{(Q_G - Q_R')/[AB + k(Q_G - Q_R')]\} \times BD + Q_R' \quad (17)$$

The number of effective stops in each cycle is given by

$$S = \left(Q_T \times \{AB + d_A' - (1/2)[(V/a) - d_A]\}\right)$$
$$\div (AB + d_A')\ (V/a) - d_A \geqslant 0 \quad (18)$$

Again, after the desired number of cycles, the final statistics may be collected by use of the calculations that follow Equation 8.

This survey method also enables calculation of the delay to pedestrians who use the intersection. The first step is to determine in which phase, or phases, pedestrian movements can occur. Assume for the moment that pedestrians on one crossing can only cross during phase A. Then, any pedestrian who arrives during phase A may cross without delay. Any pedestrian, however, who arrives during any of the other phases must wait for phase A before crossing. Pedestrian delay as a function of time of arrival is shown in Figure 12.

Assuming that pedestrians arrive randomly during the entire cycle, the average delay to pedestrians can be found by using logic similar to that used in the analysis of vehicle delay and in Figure 6. Similarly, by using the signal timings recorded in the survey as the basic input, the variability in pedestrian delay can be found by analogy with Figure 7. Delay to various pedestrian groups can be found simply by determining the phase in which they can cross and structuring the analysis program to suit. No extra data need be recorded to obtain pedestrian delay; it is simply a matter of a different analysis. It should be noted that, although the equations developed appear to be rather involved, the traffic surveyor need not be concerned with their complexities. It is a relatively simple matter to write a computer program to perform these calculations with very straightforward and easily collected input data. In fact, the analysis can be performed, if desired, on a handheld, programmable calculator, and the data can be entered directly in the field. This technique yields an instant analysis of the data.

As in most survey procedures, there is a trade-off between accuracy, complexity of analysis, complexity of data collection, and survey cost. This method has been developed with the objective of obtaining a relatively low-cost survey technique that is easily implemented in the field. At the same time, the results should be of particular relevance to bus priority intersections. In this respect, the measurement of variability in delay was considered to be an essential feature of the technique. It should be noted, however, that this survey does not give details of bus operations and that a separate survey is necessary to obtain that information. This survey gives information on the effect of bus priority on other traffic that uses the intersection.

FIELD TECHNIQUE

As mentioned earlier, one of the prime considerations in the design of this survey was that it should be easily implemented in the field. To this end, only four items of data are recorded in any one cycle on each approach to the intersection. An example of the survey form used in the field is shown in Figure 13. At the start of the green phase, the time is recorded in column A, and the queue length of stopped vehicles at that time is recorded in column B. A stopped vehicle was originally defined as one that had locked its wheels. In practice, however, it was found that many vehicles were effectively stopped although still "creeping". Some judgment is required of the observer here, and it may be advisable to have observers on different approaches observe several situations together and agree on a mutual definition before the survey begins.

A mental note is made of the last vehicle in the queue and, when the queue moves off, the progress of this vehicle is noted. At this stage, a point of clarification is needed. If one is observing a single lane of traffic, the above routine is straightforward. But if one is observing two or more lanes, how is the last vehicle in the queue defined? Is it the last vehicle in the longest queue or not? Ideally, if there are Q vehicles observed in the queue, then the end of the queue crosses the stopline when the Q th vehicle crosses the stopline independent of whether or not that particular vehicle was in the actual queue or not. Since it is sometimes difficult to count vehicles that cross the stopline, a simpler technique is used. In this, a representative end-of-queue vehicle is defined and its progress is noted. For two lanes of traffic, a vehicle in the longer queue is selected that is halfway between the ends of the long and short queues. This should then approximate the Q th vehicle in the queue. For more than two lanes, a vehicle is selected that represents the weighted average of the end of the queues. Observers report little difficulty in making this selection over three lanes.

If this representative end-of-queue vehicle crosses the stopline before the signal changes back to red, the time at which it crosses the stopline is recorded in column C. The time at which the signal changes back to red is recorded in column D (Column E is, in this case, left blank).

If this representative vehicle does not cross the stopline before the lights change to red, the time at which the lights change to red is recorded in column D and the number of vehicles in front of, and including, this vehicle when the new queue forms is recorded in column E (Column C is, in this case, left blank). This process is repeated for every cycle in the survey period.

Personnel requirements for this survey are minimal. It was found that one relatively inexperienced observer could record data on an approach to an intersection where there are low to medium rates of flow. But when the rate of flow exceeded approximately 1000 vehicles/h on an approach, it was found to be desirable to assign an assistant to count vehicles in the queue. Data were recorded directly on the field sheet shown in Figure 13. Survey equipment consisted of a watch with a second hand, survey forms, and pencils.

For instant analysis of field data, it is possible to dispense with the field sheets and input data directly to a handheld, programmable calculator. The calculator already has the analysis program stored in memory, and data for each cycle are entered directly by way of the keyboard. At the end of each cycle—that is, during the red period—the

Figure 12. Pedestrian delay versus time of pedestrian arrival.

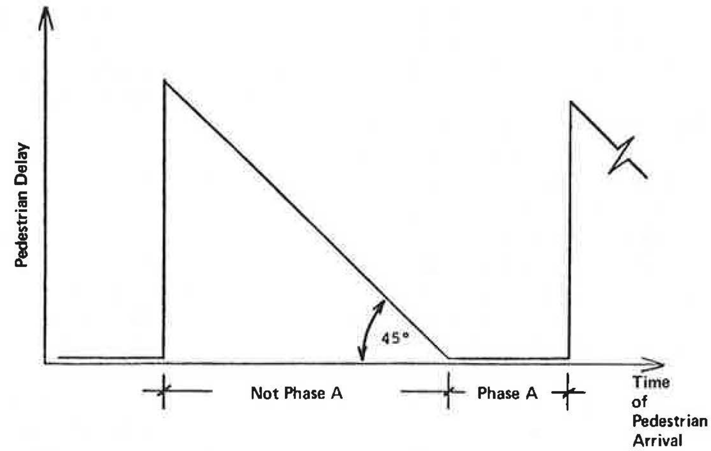


Figure 13. Example of form used in survey of queue length.

Column A			Col. B	Column C			Column D			Col. E
Time			Queue Length	Time			Time			Queue Length
Hour	Min.	Sec.		Hour	Min.	Sec.	Hour	Min.	Sec	
07	03	12	12	07	03	25	07	03	37	
07	04	21	18				07	04	40	4

analysis program is run to calculate values of Σd, Σd^2, etc., for that cycle and add them to the running total of these values. This program takes only a matter of seconds and can easily be run during the red period. Data for the next cycle are then entered as events occur and the process is repeated. At the end of the survey period, another subprogram is run to calculate the final statistics, and the analyst obtains the results in a matter of seconds after the data collection is finished.

This technique is not recommended for general, large-scale use for two reasons:

1. The data are not retrievable, and errors in data entry are not correctable. When many inexperienced observers are recording the data, such errors are inevitable.
2. The cost of equipping a complete survey team with programmable calculators is, at this time, considerable.

Future advances in programmable calculator technology will undoubtedly solve both of these problems. That is, a permanent record on input data will be possible on some form of disc or tape, and the cost of the calculators will inevitably decrease.

For the present, however, this method is recommended only for the use of the experienced professional or researcher. It enables such a person to perform instant checks on the operation of an intersection (perhaps under different control strategies). By performing 15-min sample surveys on each approach to an intersection, a general idea of intersection performance can be obtained in little more than an hour. Instead of a simple, general observation of intersection performance, a complete statistical analysis can be performed on the spot with little extra effort.

Such a technique can be used in preparation for a major survey when initial estimates of results are needed in the survey design. It can also be used when a quick decision by policy makers is needed to respond to the complaints of road users. In using the technique, one person can be dispatched to the site to obtain data and results on which an informed decision can be made. Another use—and the one that was the central objective of the overall development of this method—is the evaluation of the performance of an intersection where various configurations of bus priority signaling have been introduced. In general, the use of this method is limited only by the imagination of the user.

CONCLUSIONS

The evaluation of bus priority signals at an isolated intersection requires the use of a survey technique to measure the effects of the priority signals on other users of the intersection. The method developed and described in this paper is a low-cost, low-manpower effort that gives results that are equal, or superior, to other comparable survey methods. The survey results include vehicle flow, average and total delay, variance in vehicle delay, effective total number of stops, average pedestrian delay and variance in pedestrian delay, and a complete record of intersection signal timings.

The survey method involves recording only four variables per cycle: the start of the red period, the start of the green period, the queue length at the start of the green, and either the time at which the last vehicle in this queue crosses the stopline or the number of vehicles in this queue that are held over to the next cycle. The method is designed so that data can be recorded in the field on survey forms that are then brought back to the office for analysis or data can be keyed in directly to a programmable calculator for instant analysis in the field. The first method is recommended for large-scale surveys and the second for preliminary surveys or spot checks on intersection performance.

Although the method described here was originally developed for the evaluation of an intersection signalized for bus priority (16), the survey technique is general and can also be used for other intersections. It is recommended, however, that it not be used at intersections where the arrival of vehicles is synchronized with the timing of signals (such as at intersections along a route of coordinated signals). In that situation, a survey method similar to that of Reilly and others (6), in which the observation of queue length is done at regular intervals unrelated to signal timing, is recommended.

As a further precaution regarding the interpretation of the results of the analysis, it should be noted that, as a result of a traffic management scheme at an intersection, the delay to nonpriority vehicles at that intersection may increase. But it should not automatically be inferred from this result that, overall, nonpriority vehicles are disbenefited. The increase in delay at one intersection may be more than compensated for by a decrease in delay downstream of the intersection. It is necessary to consider at least the route effects of such TSM schemes, and it may sometimes be advisable to consider the network effects.

Gathering data in the field on route and network effects, however, may be an involved process, even when a relatively simple survey procedure, such as the one described in this paper, is used. It may be necessary to resort to simulation of the system in order to investigate these effects. To this end, I have developed an intersection simulation model to investigate the effects of various strategies of bus priority signalization. To demonstrate the generality of the survey technique described in this paper, exactly the same logic is used in the collection and analysis of data from the simulation model. Thus, a one-to-one correspondence exists between the simulation data and the field data used in validation of the model.

The survey method described here is simple, inexpensive, and flexible and generates a large array of output results from relatively few input variables. It can be used as a data collection system for the analysis of data in the office, as an instant analyzer of data in the field, or as a submodel of an intersection simulation model.

ACKNOWLEDGMENT

The work described in this paper was performed as part of a research contract for the Road Safety and Traffic Authority, Melbourne, Australia, to develop and apply an evaluation methodology for intersections with bus priority signalization. Their assistance and that of the Melbourne Metropolitan Tramways Board in the conduct of the study and the collection of the data are most appreciated. The advice and assistance of K.W. Ogden and W. Young of the Department of Civil Engineering, Monash University, are also greatly appreciated.

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Discussion

William R. Reilly, JHK and Associates, Tucson

Several comments should be made on the assumptions contained in Richardson's paper. First, the departure rates (e.g., discharge headways) of vehicles that are moving away from an intersection queue can vary considerably from the linear assumption. Where heavy pedestrian volumes are encountered with right turns, discharge headways in a right-hand lane can vary widely. The same phenomenon occurs in a left-hand lane where both left turns and through movements are allowed and where the left turns create nonlinear discharge headways for the lane.

A second situation, which is noted by Richardson and for which the assumption of linear arrivals and departures does not hold, is that of a synchronized signal system. Because platooning, or "bunching", will occur in such systems, a linear arrival pattern at a downstream signal is often not observed. Thus, the technique is not particularly applicable to lanes that exhibit platooning of either arrivals or departures.

Richardson has attempted to guide the user in conversion from a lane-by-lane survey to a total approach survey by defining a representative end-of-queue vehicle. This is a useful description since, in many cases, a measure of performance for the total approach rather than a lane-specific measure is desired.

A difficulty found in the work I performed with Gardner and Kell (6) was that, along intersection approaches that have long queues and major driveways, the number of vehicles that entered or exited the driveways could substantially alter the values for delay (by as much as 5 or 10 percent), depending on the survey technique used. Another phenomenon that does not appear to be accounted for in Richardson's method is the delay values for right-turn-on-red vehicles and the volume that crosses the stopline during a red interval.

It is these numerous "small" effects that are best captured by the more general survey technique described by my coauthors and me (6), which is based on original work by Berry and VanTil. Richardson has, however, set forth a logical and simple technique that, under certain traffic and geometric conditions, could require the use of fewer personnel than are required in the application of many other field methods.

The procedure my coauthors and I recommend for surveying delay and stops (6) does not include any measurement of signal intervals. For multi-phase-actuated equipment, this is a distinct advantage. In Richardson's method, a special technique and calculation would be required to estimate delay on an intersection approach that has "protected" or "protected-permissive" left-turn phasing. In the latter case, discharge headways in the protected and then the permissive situation are usually very different. The Federal Highway Administration work Gardner, Kell, and I described (6) concludes that, for typical field personnel, any field method that requires observation of signal phase times is less easily performed than a method that does not.

The paper by Richardson includes a section on pedestrian delay that suffers from using the same assumptions of linear (i.e., not platooned) arrivals and departures for vehicle flow. The impact of vehicle flow on pedestrians and their discharge patterns can be substantial. In addition, the actual behavior of pedestrians is often distinct from the behavior implied as a result of knowledge of signal timing and phasing. At busy intersections, it could be difficult to distinguish pedestrians who are queuing for a given crossing. Until a more explicit set of definitions and field procedures is available and the concept is validated by field data, the use of the method for studying pedestrian delay appears undesirable.

The inclusion of comments on air pollution and bus priority systems tends to distract from the central presentation of a field survey technique. It is suggested that a short user-oriented set of explicit instructions be set forth and that this include an example calculation. In this way, the reader (user) can better follow the technique. It would also be useful to check the model, and especially its assumption of linearity for arrivals and departures, against a set of field data at several intersections.

Author's Closure

The discussion by Reilly raises some important points about the survey method, especially in relation to the lack of field validation. Several issues he raises, however, require some clarification.

First, Reilly mentions the problems involved in the assumptions made for arrival and departure rates. It is true that an asssumption is made that departure rates are linear (or, more correctly, displaced linear to account for an initial start-up period). But it should also be noted that this linearity assumption can vary from cycle to cycle; that is, the assumed constant departure headway is obtained, in each cycle, by dividing the time taken to clear the queue (minus the start-up period) by the number of vehicles in the queue. Thus, if a significant interruption to departing vehicles occurs in any cycle, this is reflected in the higher than average departure headway. In this way, the departure rate accounts for such occurrences as pedestrians or opposing vehicles that may hinder the discharge process.

It is also true that linear arrival rates are assumed in each cycle (though with a variable average arrival rate in each cycle) and that the existence of synchronized platooning—that is, platoons synchronized with the signal phasing—will significantly deviate from this assumption. It should be noted, however, that the platoons must be synchronized with the signals before the survey method becomes inappropriate. Unsynchronized platoons will, on the average, have no significant effect on the survey results.

Reilly's comments on the measurement of pedestrian delay also need some clarification. He states that the method suffers here from use of the same assumptions of linearity for arrivals and departures. This, in fact, is not true. Pedestrian departures are instantaneous; that is, pedestrians leave the curb as soon as their light turns green. Pedestrian arrivals are assumed to be random rather than linear. Although it is realized that pedestrian arrivals may in fact be grouped, the assumption of randomness is satisfactory if it is assumed that pedestrian groups arrive randomly.

Reilly's comment about the actual behavior of pedestrians is of more significance. It is a well-documented fact that pedestrians do not always comply with the instructions given by signals. It has also been observed that pedestrian compliance with signals decreases as the signals cause greater delay to the pedestrians who are waiting to cross. Thus, if one calculates from the survey method that pedestrian delays are very high, it does not necessarily mean that the pedestrians are actually suffering this delay. Many will already have taken a chance and crossed the road before the light turned green. So pedestrian delay is really a combined measure of delay and risk. The greater the calculated delay is, the greater is the actual delay and risk. Either way, calculated delay is a useful measure of pedestrian signal performance.

Most of Reilly's remaining comments can be related to the purpose for which the survey method was developed. He states, "The inclusion of comments on air pollution and bus priority systems tends to distract from the central presentation. . . ." On the contrary, comments on bus priority are central to the presentation since the survey method was designed to pick up features of a bus priority intersection that could not be accounted for by other survey methods. The method was therefore designed specifically to measure (a) signal phasing and timing, which would be drastically modified by bus priority demands; (b) the variability of delay; and (c) stopped delay and effective stops, which could then be used to calculate energy consumption and air pollution emissions.

The three most important areas of further research that have emerged as a result of Reilly's comments and research conducted since the writing of this paper are the following:

1. Full field validation of the method by a comparison of field results with measures obtained from a filmed record of intersection operation,
2. Combination of Reilly's data collection method with the method of analysis presented in this paper, and
3. Development of theoretical and empirical interrelationships among various measures of intersection performance.