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Capacity and Measurement of Effectiveness

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Abridgment

Testing of the Tapeswitch System for Determining Vehicle Speed and Lateral Placement

Bernard F. Byrne, West Virginia University
Robert R. Roberts, University of South Carolina
L. Ellis King and Ronald G. Arbogast, Wayne State University

This research evaluated the use of a pressure-sensitive electrical strip switch, or tapeswitch detector, to record vehicle speed and lateral placement simultaneously. Tapeswitch can be cut to any length and used as a vehicle detector as a pneumatic tube is used. A series of tapeswitch detectors, with the appropriate electronics and recording equipment, determines the speed and lateral placement of a vehicle.

The tapeswitch system, used as part of the Bridge Shoulder Width Study at West Virginia University (WVU) (1), tested vehicle reactions (speed and lateral placement) to changes in bridge shoulder width and type of barrier.

As a part of the research, three smaller studies were undertaken to evaluate the precision and accuracy of the tapeswitch system. The first of these compared the tapeswitch system with time-lapse photography to determine lateral placement and with radar to determine speed. The second study used an instrumented vehicle equipped with a fifth wheel to determine speed when the vehicle passes over a tapeswitch and a fine powder to determine placement. The final study was a theoretical study involving error analysis.

The tapeswitch detectors are installed as a trap, consisting of three detectors: Two are installed perpendicular to the direction of travel 5.49 m (18 ft) apart, and one is installed in the middle at 45 deg to the direction of travel and extending 1.83 m (6 ft) into the roadway so that traffic in only one lane is recorded.

FORMULA FOR SPEED AND LATERAL PLACEMENT

Figure 1 shows the tapeswitch trap being used to measure vehicle speed and placement. The times at which vehicles cross tapeswitches 1, 2, and 3 are t_1, t_2, and t_3. The time to transverse is t_D for distance D and t_L for distance L. These are related as follows:

$$t_D = t_2 - t_1 \qquad (1)$$

$$t_L = t_3 - t_1 \qquad (2)$$

From these times the speed, V, and the placement, P, may be calculated.

$$V = L/t_L \qquad (3)$$

$$P = (Lt_D/t_L - R) \tan \theta \qquad (4)$$

The values for L, D, and R as used in the study are L = 5.49 m (18 ft), R = 1.83 m (6 ft), and θ = 45 deg.

COMPARISON WITH TIME-LAPSE PHOTOGRAPHY

The tapeswitch technique and time-lapse photography technique were evaluated simultaneously under three separate test conditions for determining vehicle lateral placement and speed:

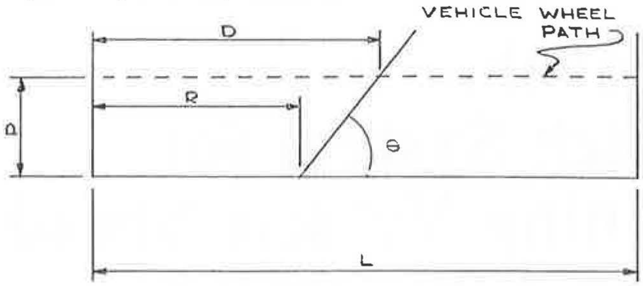
1. WVU Coliseum parking lot (L = 1.83 m, R = 0 m, θ = 45 deg);
2. WVU Coliseum parking lot (L = 5.49 m, R = 1.83 m, θ = 45 deg); and
3. I-79 field test (L = 5.49 m, R = 1.83 m, θ = 45 deg).

STATISTICAL ANALYSIS OF RESULTS

The data collected for the three study conditions were compared by statistical methods. The significance test used for the statistical analysis of the data collected for this research involves the Smith-Satterthwaite t′ statistic (2). The comparisons of the mean lateral placements for all cases show no significant differences at the 5 percent level. However, the comparison of means between vehicle speeds for all cases showed significant differences at the 5 percent level. Further, the data tended to indicate a systematic error in the radar speed measurements.

Publication of this paper sponsored by Committee on Highway Capacity and Quality of Service.

Figure 1. Tapeswitch trap dimensions.



SECOND FIELD TEST OF THE TAPESWITCH SYSTEM

The second field test used an instrumented test vehicle with a towed fifth wheel in an isolated section of a large parking lot. Approximately 15 runs were made over the trip for each 16.1-km/h (10-mph) increment between 16.1 and 96.5 km/h (10 and 60 mph). Also vehicle placement varied from as near the shoulder line as possible to the maximum limit of the tapeswitch. The speeds for each run were recorded both in the vehicle and at the tapeswitch printer. The stationary recorder also measured and recorded the vehicle placement on the pavement; a fine powder was sprinkled along the tapeswitch prior to each run, and thus distance from a reference point to the tire track was measured.

A total of 118 test runs were performed. The measured field values obtained for speed and placement were designated "actual," and the values obtained from the tapeswitch system "theoretical." A linear regression equation was thus developed to relate the actual speed and placement to the theoretical speed and placement by using a linear model, which was indicated from a graphical analysis of the data. The models developed for the theoretical speed and placement using regression analysis are

$$V_t = 0.6148 + 1.0056 V_A \qquad R_V^2 = 0.999\,01 \tag{5}$$

and

$$P_t = 0.043\,08 + 0.991\,61\,P_A \qquad R_P^2 = 0.996\,47 \tag{6}$$

where

V_t = theoretical speed,
V_A = actual speed,
P_t = theoretical placement,
P_A = actual placement, and
R^2 = coefficient of simple determination.

The R^2 values of 0.999 and 0.995 for calculated speed and placement respectively indicate a good fit of the data. Tests to determine whether the coefficients b_0 and b_1 differ significantly from zero and one respectively indicated that the constant term for both equations is significantly different from zero at the 5 percent level and that the regression coefficient on the actual values is not significantly different from one. If there were perfect agreement between the two methods of determining speed and placement, the constant term would be zero and the regression coefficient on the actual values would be one. It is highly likely that the difference is due to a calibration constant term in the placement equation and a constant error introduced by the fifth wheel.

ERROR ANALYSIS

The error in velocity measurement and lateral placement is found by using the formulas developed for the tapeswitch trap.

$$\Delta V = -L(\Delta t/t_L^2) = -(\Delta t V^2)/L \quad \text{if } \Delta t << t_L \tag{7}$$

and

$$\Delta P = L \tan\theta\,[(t_D + t_L)/t_L] = (1 + D/L)\,V \tan\theta\,\Delta t \tag{8}$$

where

ΔT = error in time,
ΔV = error in speed,
ΔP = error in placement,
ΔL = error in end tapeswitch location,
$\Delta\theta$ = error in middle tapeswitch angle, and
ΔR = error in middle tapeswitch location.

The accuracy of the timing equipment was tested by using a switch-tripping mechanism with relays. Twenty-five comparisons were made by using the testing device. The difference was either 0 or -0.1 ms. Therefore, the error associated with the field equipment was assumed to be ±0.1 ms.

Two sources of errors resulting from errors in tapeswitch placement are considered: The errors in the distances L and R and the errors in the angle θ. The effect of the errors in L can be determined by considering an error of ΔL and examining its effect on speed and placement. In this case:

$$\Delta V = (\Delta L/L)V \tag{9}$$

The errors in placement can occur in two ways: errors in lateral distance between tapeswitches and errors in the angle of the diagonal tapeswitch.

$$\Delta P = \Delta R \tan\theta \tag{10}$$

for errors in R, ΔR, and

$$\Delta P = \Delta L(D/L)\tan\theta \tag{11}$$

for errors in L, ΔL.

In considerations of speed, the maximum errors to be expected under the most adverse circumstances are less than 0.8 km/h (0.5 mph). Similarly, the maximum error in placement is (60 mm) 0.2 ft. These calculations are for the most adverse combination of circumstances and rarely occur. The probable error, which is much lower, shows what can be expected. This is less than 0.4 km/h (0.25 mph) for speed and less than 15 mm (0.05 ft) for placement. The minimum level shows what can be obtained if circumstances are fortunate or if a conscientious effort at error reduction is undertaken.

CONCLUSIONS

Three different tests were run on the tapeswitch system that simultaneously records vehicle speed and lateral placement. In the first test, the tapeswitch system was compared with time-lapse photography for recording vehicle placement and a radar meter for recording speed. The tapeswitch system results are more precise for both parts of the test. This test also determined that, of the two geometric configurations tested, the extended tapeswitch system gives better results. In the second test, vehicle speed recorded by the tapeswitch system was compared with vehicle speed recorded by a towed fifth wheel. Placement was compared by using the tapeswitch system and a fine powder spread on the ground. Again, compatibility in both cases was obtained. The third test consisted of a theoretical error analysis that determined that the maximum likely error in speed was less than 0.8 km/h (0.5 mph) and error in placement was less than (61 mm) 0.2 ft. Both of these results are well within acceptable limits. Therefore, the tapeswitch system has proved to be an accurate means for obtaining vehicle speed and lateral placement simultaneously.

ACKNOWLEDGMENTS

This study was sponsored by the West Virginia Department of Highways in cooperation with the Federal Highway Administration. The views given in the paper are ours, and we are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policy of the Federal Highway Administration, U.S. Department of Transportation, or the West Virginia Department of Highways. This paper does not constitute a standard, specification, or regulation.

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Speeds and Service on Multilane Upgrades

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This paper presents a sample of design guides for operating speeds and service levels on grades for one-way, multilane traffic including trucks. The design guides are based on computer simulation supported by field data. The guides and supplementary charts enable the user to account for truck populations with different performance characteristics. The paper also discusses traffic characteristics and comfort and safety in upgrade flows. A procedure is described for using the reported results to predict peaking characteristics in upgrade flows.

Trucks on upgrades of multilane facilities reduce capacity and service level. However, no well-established data base or comprehensive model is available to quantify these effects. The 1965 Highway Capacity Manual provides multilane truck equivalence values that are apparently based on some informal traffic observations on multilane facilities plus field studies and equivalence values for two-lane, two-way highways. The manual also presents a conceptual model that is quantified with a small amount of data. This paper discusses the use of field data and analyses to better quantify the effects of trucks in upgrade flows on multilane facilities (1, 2, 3, 4, 5, 6).

METHODS

A microscopic simulation model for unidirectional flow on two or three lanes was developed and computerized. The model was adjusted and validated by using data from the literature and field data collected by ground observers at 22 locations on six grades. In addition, a small amount of data were collected by aerial photography on one grade.

The model duplicates multilane flow features in situations ranging from free-flowing to congested conditions in level terrain, on grades, in the transition regions at grade feet and crests, and at climbing lane additions and drops. Characteristics tested include distribution of flow to lanes, lane change rates, time headways, and speeds. Numerous traffic characteristics are duplicated in the wide variety of terrains because of model logic and without a priori judgments imposed through input.

The program consists of about 8000 statements, is written in FORTRAN IV (except for a small routine in assembly language), and on the CDC 6000 computers requires about 32 000 words of core in addition to the system. User instructions and a complete description of the model, model adjustment, and validation are given in other reports (1, 2, 3, 4, 5, 6). This paper presents major results obtained with the model.

The results from the model tend to confirm a basic postulate of traffic engineering, namely, that the operating speed (and the passenger vehicle average speed) plotted against flow rate exhibits a characteristic shape. And the capacity flow is diminished by slow trucks. Further, the capacity is a nonlinear function of both the percentage of trucks and the local speeds of the trucks.

The design charts described in the next section are based on numerous model results that have been assembled by using the characteristic relations between speed and flow. The curve of operating speed versus percentage of capacity was established first. Then the model was exercised with a variety of flow rates, percentages of trucks, and truck populations on grades. Each case provided an operating speed that was used to read the associated percentage of capacity. The capacity was then estimated as the flow rate divided by the percentage of capacity. The results were assembled into design chart sets.

DESIGN CHART SETS

Design guide chart sets were assembled from the results of numerous simulation model runs. Each chart set consists of two figures that can be used to estimate speed and service for short time periods. Figures 1 and 2 constitute a set for two upgrade lanes on a facility with 121-km/h (75-mph) design speed. Figure 2 shows added lines associated with two examples that are described.

In the first example it is desired to estimate the freeway service level and operating speed on a 2 percent sustained grade with 10 percent trucks in a mixed flow of 1800 vehicles/h. An initial point is located on Figure 2

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at the intersection of the 2 percent grade line and the 10 percent truck line. From the initial point, the horizontal line (1-1) is followed to the intersection with 1800 vehicles/h, and the fan of lines is followed (along 1-2) to the scale for percentage of estimated capacity, which is read as 56. At 56 percent, the service level is C′ and the operating speed is 97 km/h (60 mph). (Service levels are primed to remind the user that the service level depends on operating speed and percentage of capacity. Comfort and safety on grades may not equal that in level terrain.)

In the second example the maximum flow for service level C′ on a rural multilane highway is sought for a 4

Figure 1. Operating speed versus percentage of capacity.

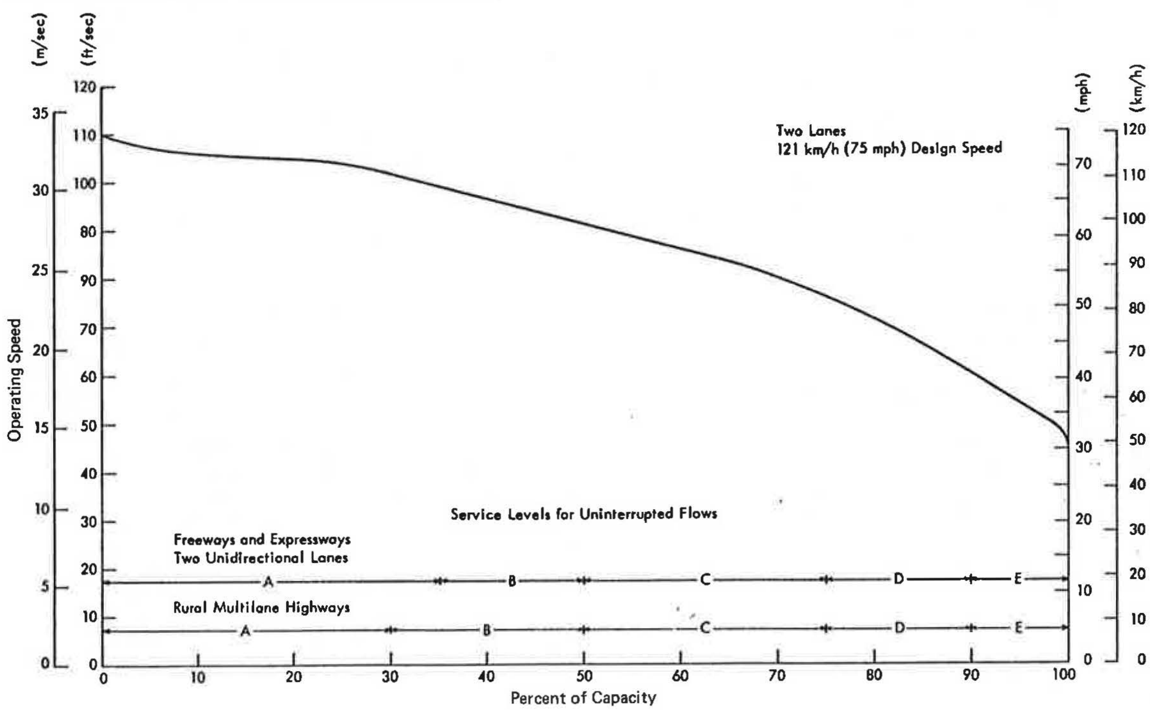
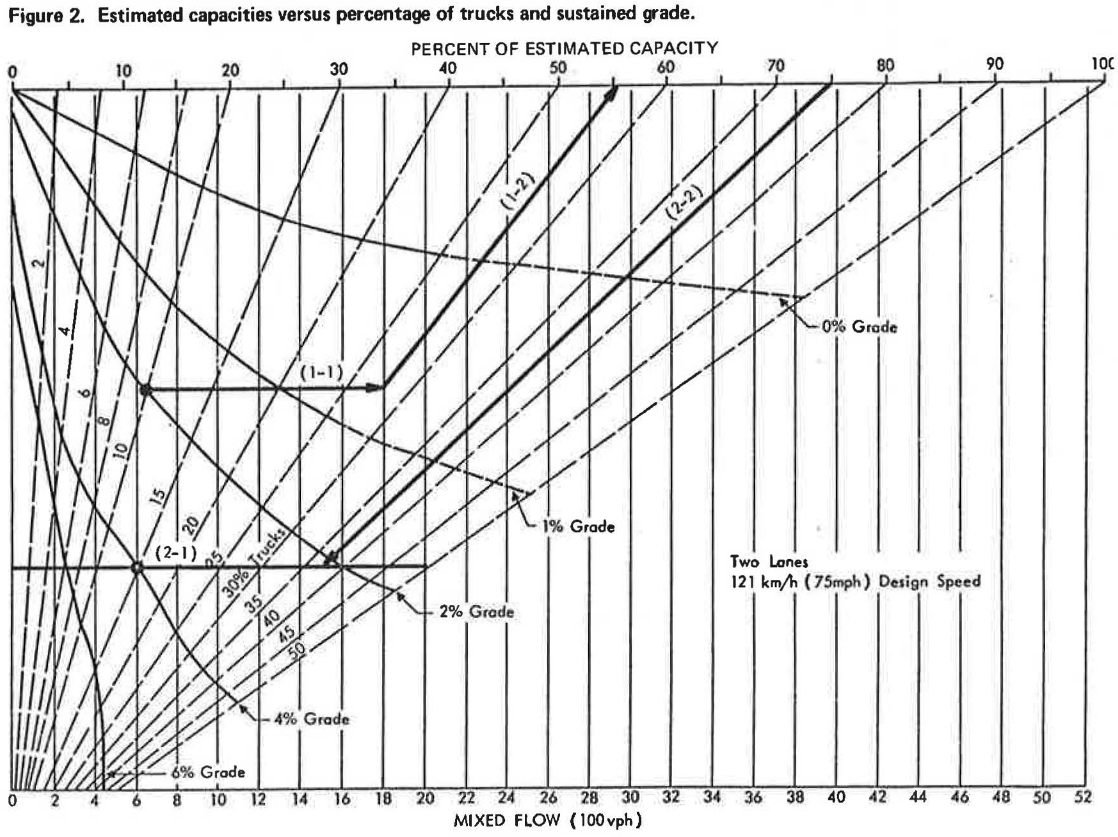


Figure 2. Estimated capacities versus percentage of trucks and sustained grade.



percent sustained grade on which the flow will contain 15 percent trucks. The intersection of the 4 percent grade line and the 15 percent truck line is located on Figure 2 and a horizontal line is passed through that point (line 2-1). Figure 1 shows that the upper limit for level C′ is 75 percent of capacity. Figure 2 is entered on the percentage of estimated capacity scale at 75 percent. The fan of lines is followed (along line 2-2) to the intersection with line 2-1. At that intersection the answer is read as 1520 mixed vehicles/h.

Figure 2 and others like it are based on a simple concept that involves the horizontal line passing through the intersection of curves for the percentages of grade and trucks. Service levels A′ (if possible) through E′ are represented along the horizontal line. At the left end, the percentage of capacity is zero and the flow is zero. This is the highest possible service level. At the right of the line, where it intersects the last of the fan of lines, the mixed flow is equal to the estimated capacity. For the second example the estimated capacity is 2020 vehicles/h. The fan of lines actually serves a dual purpose. First, individual lines in the fan identify the intersection of the given percentage of trucks with a sustained grade line. Second, from any point on a horizontal line the fan can be used as a guide to the scale for percentage of estimated capacity.

In the next section the design chart sets are applied to grade feet and crests and to rolling terrain. However, the charts apply to a specific truck population and provide an estimate of the most likely flow conditions during short time periods (2 to 3 min).

VEHICLE POPULATION AND WEIGHT FACTORS

The performance characteristics of the vehicles, especially the trucks, influence the service levels and capacities in on-grade flows. The acceleration and speed capabilities of the trucks in a flow may be as important as the number of trucks. This section presents procedures that can be used to account for the variations in truck populations on different facilities.

The design charts are based on passenger vehicle and truck populations with the characteristics in Tables 1 and 2. The truck population given in Table 2 contains a relatively large proportion of low-performance trucks. The design charts, based on this reference population, will usually provide conservative results (low service levels). A national average truck population, based on data reported by Wright and Tignor (7), is estimated as 26 percent for index 7, 40 percent for index 8, 24 percent for index 9, and 10 percent for index 10.

Because the user may have to contend with a different population, results from the simulation model were used to derive weight factors for adjusting other truck populations to the reference population given in Table 2. The weight factors apply to these four truck types as shown in the following equation.

$$\text{Percentage of reference trucks} = (100/F)(3.16 f_{10} + 1.41 f_9 + 0.14 f_8 + 0.06 f_7) \quad (1)$$

where

F = total flow rate of mixed vehicles,
f_{10} = flow rate of Index No. 10 trucks, and
f_9 = flow rate of Index No. 9 trucks and so on.

Percentage of reference trucks is in terms of the reference population defined in Table 2. The equation can also be expressed purely in terms of percentages.

$$\text{Percentage of reference trucks} = P_T(3.16 p_{10} + 1.41 p_9 + 0.14 p_8 + 0.06 p_7) \quad (2)$$

where

P_T = total percentage of trucks observed, and
P_{10} = ratio of Index No. 10 trucks to total trucks and so on.

The weight factors are internally consistent; when the weight factors are applied to the reference population, the percentage of reference trucks equals the percentage based on direct counts.

If trucks other than those given in Table 2 are encountered, their weight factors can be approximated by using their speed differences from the slowest observed trucks. (This is an approximation because the speed differences between trucks are not exactly the same on sustained grades of 2, 4, and 6 percent.) The weight factors as a function of speed differences are shown in Figure 3. The slowest truck in a sample (3-min sample) is assigned the weight factor 3.16, or 3.0 in the linear approximation.

Figure 3 provides a means for assigning weight factors to trucks without their first being equated to a truck type. In addition, the strong sensitivity of flow characteristics to the slowest truck in a sample suggests that design chart information can be expressed as a function of the speed of the slowest truck. This representation is shown in Figure 4 as estimated capacity versus the speed of the slowest truck and the percentage of reference trucks.

Figure 4 can be used in lieu of Figure 2 to estimate capacity so that percentage of capacity can be calculated for a design or projected flow. Then, estimates for the service level and operating speed can be read from Figure 1. Figure 4 also can be used for capacity estimates on grades that were not explicitly simulated. The 1 percent grade line in Figure 2 was obtained this way. Figure 4, in conjunction with Figures 1 and 3, can also be used to estimate flow conditions in the foot and crest transition regions when the speeds of the truck sample are known or estimated for these regions.

Figure 4 and similar figures have been used to extrapolate to large truck percentages. The simulation results extend to 20 percent or 30 percent trucks for the grade and lane combinations.

The flow characteristics in rolling terrain should be equivalent to a sequence of foot and crest transition flows. As an example, consider the influence of a short grade on a facility with two upgrade lanes. The alignment has a sag vertical curve at the foot followed by 122 m (400 ft) of 4 percent grade. It is estimated that the truck population, which constitutes 17.5 percent of the peak-hour flow, is slowed to the minimum speeds given in Table 3. When the weight factors are applied, the percentage of reference trucks is

$$\begin{aligned}\text{Percentage of reference trucks} &= 17.5[(0.05)(3.16) + (0.08)(2.72) \\ &\quad + (0.21)(1.61) + (0.33)(0.92) \\ &\quad + (0.33)(0.37)] \\ &= 19.9\end{aligned} \quad (3)$$

The estimated capacity is read from Figure 4 at 19.9 percent trucks and 13.7 m/s (45 ft/s) for the slowest truck. The estimated capacity is 3420 mixed vehicles/h. Figure 1 may be used to estimate service level. Service near the crest of the short grade will fall below level C′ if mixed flow exceeds 75 percent of 3420 or 2562 vehicles/h.

In the above example, it must be recognized that the service would be depressed over a short section of highway. Variations over time and grade length are discussed later.

An earlier report (1) includes figures similar to Figure 4 for two and three lanes with design speeds of 105, 113, and 121 km/h (65, 70, and 75 mph).

PRECAUTIONS IN THE USE OF DESIGN CHARTS

The design charts are based on a truck population with a large percentage of low-performance vehicles, and the basic curves were drawn conservatively. However, the charts are based on flows during short time periods. They do not include provision for peaking or variance during a design hour. Also, the user should recognize that the charts provide estimates for traffic conditions in relatively short sections of highway, 300 to 600 m (1000 to 2000 ft). Consequently, the truck speeds used are the local speeds and not speeds averaged over the entire grade.

The simulation results indicate that the estimated capacities are not "practical capacities." Temporary local congestions can occur in the on-grade flows over a wide range of percentages of estimated capacity. When 90 percent of estimated capacity is approached, temporary local congestions are almost certain to occur. When flow exceeds 90 or 95 percent of implied capacity, the flow is vulnerable to breakdown. It appears that breakdown on a 2 percent grade will occur in a fashion similar to flows in level terrain with nearly uniform high densities and low speeds. On the 4 and 6 percent grades with trucks present, the maximum flows appear to occur as a sequence of congested and flowing conditions on the grade. Flow breakdown on these steeper grades is probably not a smoothly spreading phenomenon. Instead, if the spots of congestion grow, the storage space on the grade becomes progressively filled with congestion queues.

Some of the poorest service levels simulated occurred on the 4 percent grades with two lanes upgrade. The service was depressed in local sections on the grade by a flow feature that might be described as persistent congestion. This congestion was triggered by a single event or a sequence of disruptive events, such as a truck passing a platoon of other trucks. In one case a conservative driver followed the truck performing the pass so that the queue was slow to accelerate when the truck returned to the right lane. In each case a queue 180 to 300 m (600 to 1000 ft) long built up in the median lane. This local spot of congestion did not dissipate; it followed the slow truck up the grade. A flow of higher speed vehicles went through the congestion (with delay), but the congested queue remained with slowly varying length.

In some applications the third upgrade lane may be a climbing lane, which is added near the grade foot and dropped near the crest. The full benefits of the third lane are not realized in the first 240 to 300 m (800 to 1000 ft). Likewise, the benefits are not fully retained during the last 300 to 425 m (1000 to 1400 ft) of the added lane. For volume/capacity ratios of 35 to 85 percent, the operating speeds will be depressed about 2.4 m/s

Table 1. Characteristics of passenger vehicle population.

Simulation Index No.[a]	Percentage in Passenger Vehicle Population	Length (m)	Max Acceleration[b] (m/s^2)	Max Speed[b] (km/h)
1	49.80	5.5	4.5	161.3
3	45.13	5.5	3.6	142.6
5	5.07	5.5	2.5	110.9

Note: 1 m = 3.28 ft; 1 km/h = 0.6 mph.
[a]Indexes 2 and 4 were not used. [b]On zero grade at sea level conditions.

Table 2. Characteristics of reference truck population.

Index No.	Weight Power Represented (kg/kW)	Percentage in Commercial Truck Population	Length (m)	Max Acceleration[a] (m/s)	Performance-Limited Steady Speed (m/s) by Grade[b] 0%	1%	2%	3%	4%	5%	6%	7%
7	56	13.5	7.6	3.96	33.1	30.9	28.9	27.1	25.4	23.8	22.4	21.0
8	112	36.5	12.2	2.82	29.4	25.6	22.4	19.8	17.6	15.7	14.1	12.6
9	187 to 261	36.5	15.2	1.73	25.0	20.1	16.4	13.6	11.2	9.5	8.0	6.7
10	>261	13.5	18.3	1.16	18.7	12.8	9.3	7.0	5.3	4.1	3.2	2.4

Note: 1 kg = 2.2 lb; 1 kW = 1.34 hp; 1 m = 3.28 ft.
[a]On zero grade at sea level conditions. [b]At sea level conditions.

Figure 3. Weight factors for trucks versus speed above lowest truck speed.

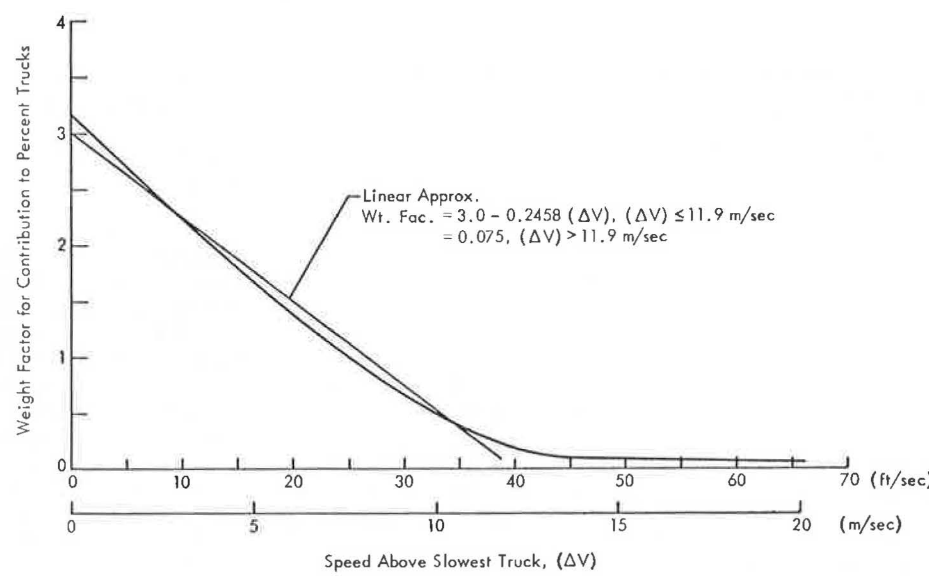


Figure 4. Estimated capacity versus speed of slowest truck and percentage of reference trucks.

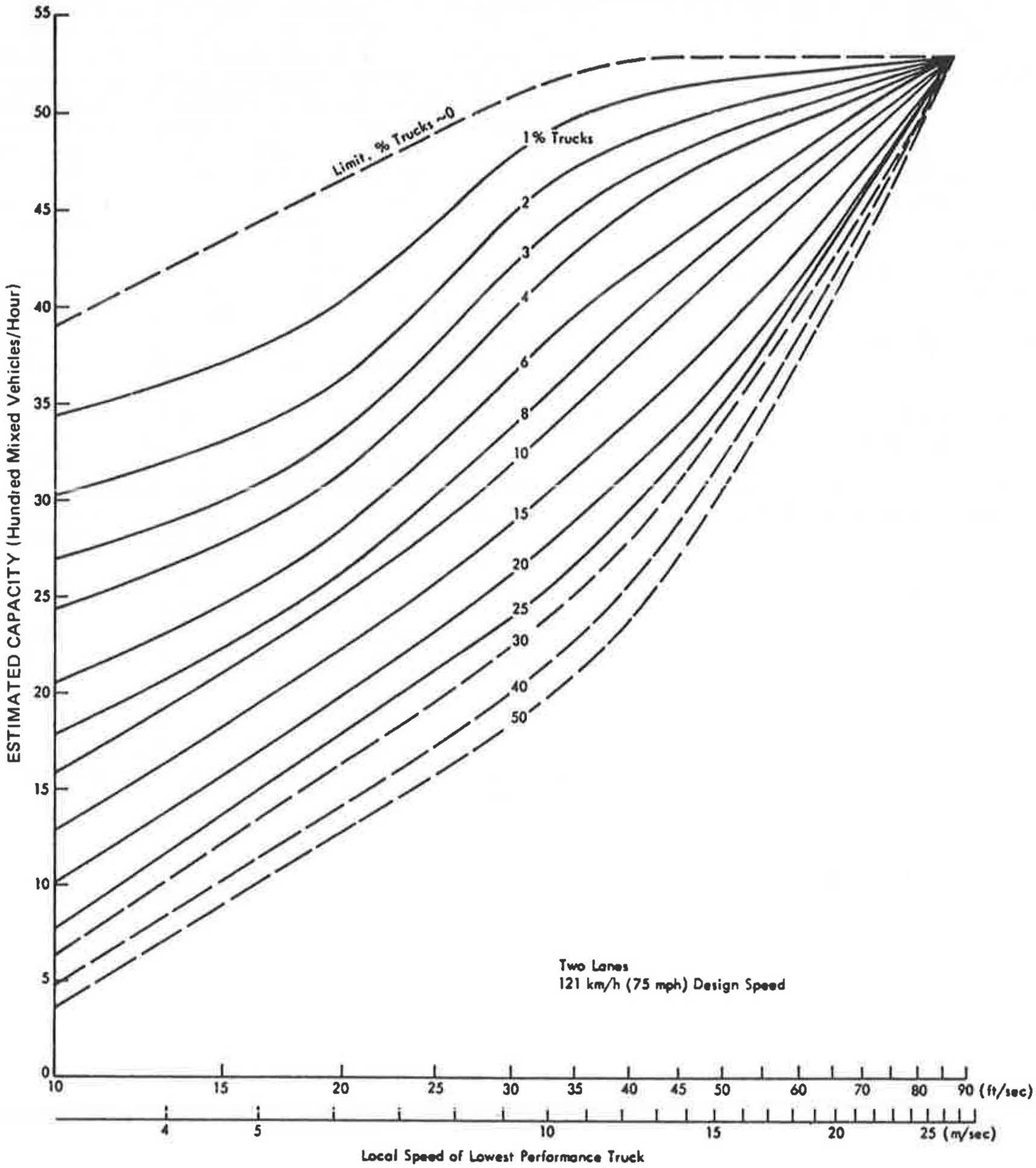


Table 3. Minimum spot speeds of trucks on a short grade and associated weight factors.

Percentage of Truck Population	Minimum Speed (m/s)	Speed Above Slowest (m/s)	Weight Factor (From Figure 3)
5	13.7	0	3.16
8	15.2	1.5	2.72
21	18.9	5.2	1.61
33	21.6	7.9	0.92
33	24.4	10.7	0.37

Note: 1 m = 3.28 ft.

(8 ft/s) after the lane addition and before the drop. The underlying traffic behavior is observed in the model and at field sites.

The reader is also cautioned that the simulation model was exercised for cases extending to 20 to 30 percent trucks. The extrapolation to 50 percent trucks was made when the results from individual cases were summarized to the design charts. The extrapolations that are subject to logical constraints are included in the design charts to emphasize the nonlinearities suggested by available results.

VARIATIONS OF FLOW CHARACTERISTICS WITH TIME AND LENGTH

The data collected on grades and the observation of the flows suggest that 2 to 3 min is a suitable time period for relating the flow characteristics on short sections to the flow rate and vehicle population. Longer time periods will average over characteristics that may be noticeably different. This short period is at variance with the hourly rates and volumes that are normally used in design or evaluation. However, the importance of short-term demands has been recognized for flows on freeways and expressways, and the 5-min interval peak has been employed. Peak-hour factors are used to account for the mean maximum demand during 5-min periods of peak hours. When the period is shortened to 3 min, the peaking will be slightly more severe. However, on grades (especially on sustained grades) there are additional sources of variance, some of which may be more important than the increased peaking in flow rate.

The design-hour volume for a facility, together with the percentage of trucks, may be the basis for design or evaluation. A peaking factor may be used to account for total flow variations and to estimate the mean maximum flow rate. On a sustained grade, however, the variation of truck flow rates between 3-min intervals may be the source of equal or greater variation in traffic characteristics. In addition, the samples of trucks that arrive in individual 3-min periods may have performance capabilities that are different from those of the truck population sampled during long periods. The simulation results and the comparisons with field data indicate that the size and character of the truck sample have a strong effect on the short-period flow characteristics. Neither of these types of sample-to-sample variations has a pronounced effect on flow characteristics in level terrain.

An additional source of variation for the on-grade flow arises from the presence or absence of disruptive events in the flow. The disruptive events are usually associated with truck-passing-truck maneuvers. This means that the flow characteristics might vary noticeably between different 3-min samples even though each contained exactly the same set of trucks.

All the features discussed above increase the variance of operating speeds measured during 3-min periods on a short section, 300 m (1000 ft), of a grade. When the flow is examined along an extended grade, the same flow features cause a space-wise variation in operating conditions. The space-wise variation is encountered by passenger vehicle drivers who ascend an extended grade in flows that are significantly influenced by slow trucks and who may be forced to make more speed changes and lane changes than would be required for the same overall operating speed in level terrain. As a result the safety and comfort aspects of service on a grade may not equal those in level terrain even when overall operating speeds are equal.

Additional length for a sustained grade appears to have two deleterious effects. First, it increases the likelihood that a region of severely depressed service will exist on the grade at any given time. Second, when spots of persistent congestion arise, they affect the flow for a time proportional to the remaining distance to the crest.

The design charts can be used with an hourly flow rate; the truck population is averaged for the hour to give an estimate of the average operating conditions for the hour. For design and traffic engineering it is more informative to know the distribution of operating conditions. Obviously, the low-service points are most important. The distributions can be constructed by using the design charts and 3-min traffic samples generated with a combination of probabilistic and stochastic techniques.

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Passenger Car Equivalencies of Trucks, Buses, and Recreational Vehicles for Two-Lane Rural Highways

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Passenger car equivalents or adjustment factors for trucks, buses, and recreational vehicles are often required in carrying out highway capacity calculations. This paper presents, in part, the results of a research project into the effect that recreational vehicles have on highway capacity. Described are the underlying methods referred to in the 1965 Highway Capacity Manual for determining passenger car equivalents, the methods used for developing equivalencies for recreational vehicles, and the use of the new equivalencies for typical highway capacity computations. Sensitivity testing of a recreational vehicle simulator model is discussed. Results of the sensitivity testing, which was extended to include highway capacity computations, strongly indicate that the present passenger car equivalent speed curves and adjustment factors in the 1965 Highway Capacity Manual require further refinement and updating, particularly at slower speeds. This paper estimates their correct placement by applying basic traffic engineering relationships.

During recent years there has been a phenomenal increase of recreational vehicles (i.e., travel trailers, campers, camper trucks, motorhomes, and vans) in the traffic stream. The 1965 Highway Capacity Manual (HCM) (1) does not make any provision for the effect of recreational vehicles; consequently, highway planning studies (2) have begun using adjustment factors for trucks and buses to estimate large-vehicle effect. Using such factors has not always proved to be accurate, particularly in cases where the site considered is rolling or mountainous terrain. Because little research has been done to quantify in absolute terms the effect of recreational vehicles and because most of Alberta's recreational areas are located in a mountainous region where recreational vehicles constitute as high as 30 percent of the traffic stream, a research project was initiated by Alberta Transportation and undertaken by the University of Calgary (3).

DETERMINATION OF PASSENGER CAR EQUIVALENTS

The HCM (1, p. 101) states:

> On two lane highways, passenger car equivalents of trucks are obtained relatively easily. They can be directly determined by obtaining detailed information on the speeds and headways of vehicles during various rates of flow on highways with different alignments and profiles. An average passenger car equivalent is obtained for trucks under each condition.... Passenger car equivalents can also be calculated with a high degree of accuracy from the separate speed distributions of passenger cars and trucks at any given volume level. The criterion used is the relative number of passings that would be performed per mile of highway if each vehicle continued at its normal speed for the conditions under consideration.

Passenger car equivalents (PCEs) for trucks or other large vehicles are needed because of two main factors:

1. A truck traveling more slowly than the passenger car traffic stream restricts and reduces capacity and level of service, and
2. A truck traveling at approximately the same speed as the passenger car traffic stream occupies more space because of its size and possibly requires greater stopping distance.

The first factor is prevalent on grades in rolling or mountainous terrain. For such conditions, Walker's method, which is described briefly in this paper, has been applied. This method was used to derive PCEs contained in the 1965 HCM. The second factor is more common in level terrain and the time interval (headway) method may be applied to determine the PCE.

The criteria developed by Walker are based on the number of passings or overtakings that would be performed per kilometer of highway if each vehicle continued at its normal speed for the conditions under consideration. The general case is

$$N = \sum_{i=1}^{n} \sum_{j=1}^{m} X_i Y_j \bigg/ 60\left(\frac{60}{S_{2_i}} - \frac{60}{S_{1_j}}\right) \tag{1}$$

in which N is the sum of the overtakings in terms of vehicles traveling at speed S_1 that will overtake X vehicles/h traveling at speed S_2 within 1 km (0.6 mile) of the highway when the number of vehicles traveling at speed S_1 is Y vehicles/h. The numbers of slower and faster vehicles are n and m respectively at a selected speed grouping.

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In Figure 1, for example, S_1 and S_2 are in units of kilometers per hour and the speed groupings increase in increments of 10 km/h (6.21 mph).

Equation 1 permits one to carry out computations for any particular speed distribution and any slower vehicle to arrive at a PCE (Figure 1). The final step is to calculate the ratio of passenger cars to a slower moving vehicle (truck) as follows:

$$\text{Ratio} = \frac{N/1 \text{ truck/h}}{N/100 \text{ passenger cars/h}} = \frac{3.25/1 \text{ truck/h}}{20.61/100 \text{ passenger cars/h}} \qquad (2)$$

The ratio of 15.8 is the PCE of a truck traveling 20 km/h (12.42 mph). By using speed distributions for various levels of service and equation 2 above, a PCE speed curve can be produced as shown in Figure 2.

The headway method is best suited to determine equivalencies on level terrain at low levels of service. The method is based on the concept that a truck occupies more space than a single passenger car and therefore reduces capacity. The procedure involves the measurement of the time interval (headway) between vehicles and their speed. This procedure does not consider the passing or the desire of drivers to pass as does the Walker method. The basic equation for the headway method is

$$E = (h/p - c)/t \qquad (3)$$

where

E = PCE for truck,
h = average headway for a sample of cars and trailers,
p = average headway for all-passenger-car sample,
c = proportion of cars, and

Figure 1. Matrix for determining passenger car equivalents.

No. of Vehicles X at slower speed S_2		No. of vehicles Y at faster speed S_1											Total
		0	0	2	10	19	37	22	8	2	0	Y	100
X	$\frac{1}{S_2}$ S_2	0.00911 110	0.0100 100	0.0111 90	0.0125 80	0.0143 70	0.0167 60	0.0200 50	0.0250 40	0.0333 30	0.0500 20	$\frac{1}{S_1}$ S_1	
0	0.0500 20	0.0409 -	0.0400 -	0.0390 -	0.0375 -	0.0357 -	0.0333 -	0.0300 -	0.0250 -	0.0167 -	- -		
2	0.0333 30	0.0242 -	0.0233 -	0.0222 0.0888	0.0208 0.0416	0.0290 0.7220	0.0166 1.2284	0.0133 0.5852	0.0083 0.1328	-	-		3.1732
8	0.0250 40	0.0159 -	0.0150 -	0.0139 0.2224	0.0125 1.0000	0.0107 1.6264	0.0083 2.4568	0.0050 0.8800	- -	- -	- -		6.1856
22	0.0200 50	0.0109 -	0.0100 -	0.0089 0.3916	0.0075 1.6500	0.0057 2.3826	0.0033 2.6862	- -	- -	- -	- -		7.1104
37	0.0167 60	0.0076 -	0.0067 -	0.0056 0.4144	0.0042 1.5540	0.0024 1.6872	- -	-	-	-	-		3.6556
19	0.0143 70	0.0052 -	0.0043 -	0.0032 0.1216	0.0018 0.3420	- -							0.4636
10	0.0175 80	0.0034 -	0.0025 -	0.0014 0.0280	-								0.0280
2	0.0111 90	- -	- -										
0	0.0100 100	- -											
0	0.0091 110												
													20.6164

1	0.0500 20	0.0409 -	0.0400 -	0.0390 0.0778	0.0375 0.3750	0.0357 0.6783	0.0333 1.2321	0.0300 0.6600	0.0250 0.2000	0.0167 0.0334	- -		3.2566

10 km/h = 6.21 mph

t = proportion of trucks.

For the study on the effect of recreational vehicles, the mean headway for any specific number of vehicles for each percentage of recreational vehicles was substituted in equation 3. The resulting plot is E versus volume for all ranges in recreational vehicle percentage for which data are available. The PCEs for recreational vehicles were calculated by using equation 2 in slightly modified form:

$$E_r = \frac{\{(\text{all}/\text{pp}) - [1 - (\%\ \text{recreational vehicles}/100)]\}}{\%\ \text{recreational vehicles}/100} \quad (4)$$

where

E_r = PCEs for recreational vehicles,
all = average headway for all vehicles in the traffic stream for the time interval considered, and
pp = average headway of a passenger car following a passenger car for the time interval considered.

The average PCE for 1-h intervals was 1.6 at an average volume of 1000 vehicles/h for the lane of the two-lane highway studied. A value of 1.6 was suggested for levels of service D and E.

Figure 2. Passenger car equivalent-speed curve for two-lane highways.

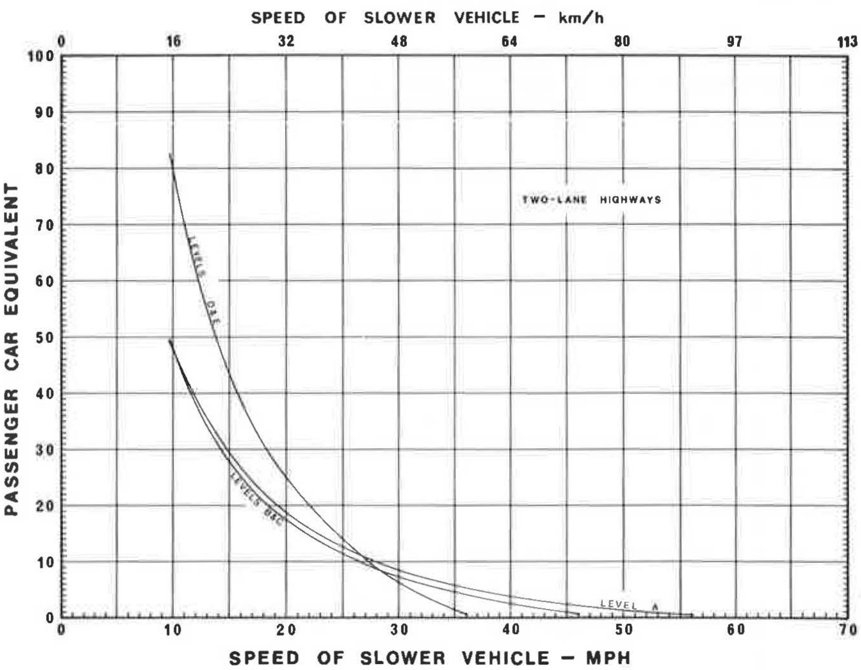


Figure 3. Average speed of recreational vehicles over entire length of grade on two-lane highways.

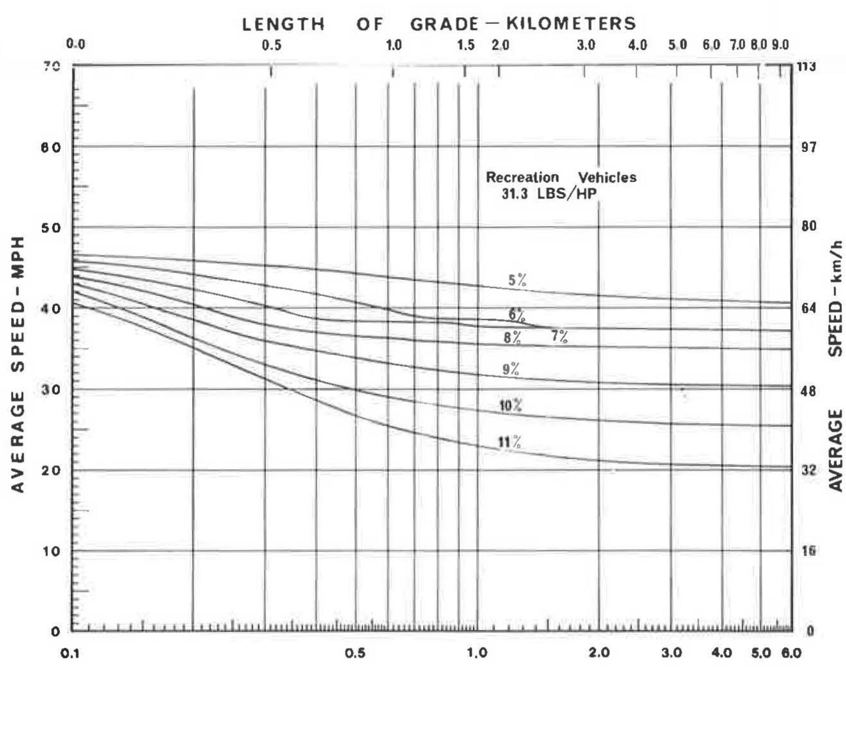


Table 1. Passenger car equivalents of trucks, buses, and recreational vehicles on two-lane highways by levels of service.

Grade (%)	Length of Grade (km)	Levels of Service A and B				Level of Service C				Levels of Service D and E			
		Trucks[a]	Buses[b]	RVs[c]	PVs[d]	Trucks	Buses	RVs	PVs	Trucks	Buses	RVs	PVs
0 to 2	All	2	2	2.3	1.0	2	2	1.6	1.0	2	2	1.6	1.0
3	0.4	5	2	2.3	1.0	3	2	1.6	1.0	2	2	1.6	1.0
	0.8	10	2	2.3	1.0	10	2	1.6	1.0	7	2	1.6	1.0
	1.2	14	2	2.3	1.0	16	2	1.6	1.0	14	2	1.6	1.0
	1.6	17	2	2.3	1.0	21	2	1.6	1.0	20	2	1.6	1.0
	2.4	19	2	2.3	1.0	25	2	1.6	1.0	26	2	1.6	1.0
	3.2	21	2	2.3	1.0	27	2	1.6	1.0	29	2	1.6	1.0
	4.8	22	2	2.3	1.0	29	2	1.6	1.0	31	2	1.6	1.0
	6.4	23	2	2.3	1.0	31	2	1.6	1.0	32	2	1.6	1.0
4	0.4	7	2	2.3	1.0	6	2	1.6	1.0	3	2	1.6	1.0
	0.8	16	2	2.3	1.0	20	2	1.6	1.0	20	2	1.6	1.0
	1.2	22	2	2.3	1.0	30	2	1.6	1.0	32	2	1.6	1.0
	1.6	26	2	2.3	1.0	35	2	1.6	1.0	39	2	1.6	1.0
	2.4	28	2	2.3	1.0	39	2	1.6	1.0	44	2	1.6	1.0
	3.2	30	2	2.3	1.0	42	2	1.6	1.0	47	2	1.6	1.0
	4.8	31	2	2.3	1.0	44	2	1.6	1.0	50	2	1.6	1.0
	6.4	32	2	2.3	1.0	46	2	1.6	1.0	52	2	1.6	1.0
5	0.4	10	4	2.3	1.0	10	3	1.6	1.0	7	2	1.6	1.0
	0.8	24	4	2.7	1.0	33	3	1.6	1.0	37	2	1.6	1.0
	1.2	29	4	2.9	1.0	42	3	1.6	1.0	47	2	1.6	1.0
	1.6	33	4	3.0	1.0	47	3	1.6	1.0	54	2	1.6	1.0
	2.4	35	4	3.2	1.0	51	3	1.6	1.0	59	2	1.6	1.0
	3.2	37	4	3.3	1.0	54	3	1.6	1.0	63	2	1.6	1.0
	4.8	39	4	3.5	1.0	56	3	1.6	1.0	66	2	1.6	1.0
	6.4	40	4	3.6	1.0	57	3	1.9	1.0	68	2	1.6	1.0
6	0.4	14	7	2.9	1.0	17	6	1.6	1.0	16	4	1.6	1.0
	0.8	33	7	3.6	1.0	47	6	2.0	1.0	54	4	1.6	1.0
	1.2	39	7	4.3	1.0	56	6	2.3	1.0	65	4	1.6	1.0
	1.6	41	7	4.4	1.0	59	6	2.5	1.0	70	4	1.6	1.0
	2.4	44	7	4.8	1.0	62	6	2.9	1.0	75	4	1.6	1.0
	3.2	46	7	4.9	1.0	65	6	3.0	1.0	80	4	1.6	1.0
	4.8	48	7	4.9	1.0	68	6	3.0	1.0	84	4	1.6	1.0
	6.4	50	7	5.0	1.0	71	6	3.1	1.0	87	4	1.6	1.0
7	0.4	22	12	3.3	1.2	32	12	2.2	1.0	35	10	1.6	1.0
	0.8	44	12	3.7	1.4	63	12	2.6	1.0	75	10	1.6	1.0
	1.2	50	12	4.6	1.5	71	12	2.7	1.0	84	10	1.6	1.0
	1.6	53	12	4.8	1.6	74	12	2.8	1.0	90	10	1.6	1.0
	2.4	56	12	4.8	1.7	79	12	2.9	1.0	95	10	1.6	1.0
	3.2	58	12	4.9	1.8	82	12	3.0	1.0	100	10	1.6	1.0
	4.8	60	12	4.9	1.9	85	12	3.0	1.0	104	10	1.6	1.0
	6.4	62	12	5.0	1.9	87	12	3.1	1.0	108	10	1.6	1.0

Note: 1 km = 0.6 mile.
[a]Values are from Highway Capacity Manual (1, Table 10.10, p. 305).
[b]Values are from Highway Capacity Manual (1, Table 10.11, p. 306).
[c]Recreational vehicles.
[d]Passenger vehicles.

Table 2. Average generalized passenger car equivalents of trucks, buses, recreational vehicles, and passenger cars on two-lane highways by terrain.

Vehicle	Level of Service	Equivalent by Terrain[a]		
		Level	Rolling	Mountainous
Truck	A	2.0	4.0	7.0
	B and C	2.2	5.0	10.0
	D and E	2.0	5.0	12.0
Bus	A	1.8	3.0	5.7
	B and C	2.0	3.4	6.0
	D and E	1.6	2.9	6.5
Recreational vehicle	A	2.2	3.2	5.0
	B and C	2.5	3.9	5.2
	D and E	1.6	3.3	5.2
Passenger vehicle	A	1.0	1.3	2.3
	B and C	1.0	1.0	2.5
	D and E	1.0	1.0	1.0

[a]Some of the values for trucks and buses have been readjusted from those given in the Highway Capacity Manual (1, Table 10.9a, p. 304).

Table 3. Relation of maximum grades to design speed on main highways.

Terrain	Design Speed (km/h)							
	50	65	80	100	105	110	120	130
Level	6	5	4	3	3	3	3	3
Rolling	7	6	5	4	4	4	4	4
Mountainous	9	8	7	6	6	5	—	—

Note: 1 km = 0.6 mile.

PASSENGER CAR EQUIVALENCIES FOR TRUCKS, BUSES, AND RECREATIONAL VEHICLES FOR USE IN HIGHWAY CAPACITY CALCULATIONS

Performance of recreational vehicles can be predicted with reasonable accuracy by a linear equation whose coefficients are related to vehicle characteristics. The vehicle characteristics considered include maximum acceleration capability, maximum speed, power, weight, rear axle and transmission gear ratios, drag coefficient, frontal area, and air-mass density. A vehicle simulation model, developed by A. D. St. John of the Midwest Research Institute and incorporating various equations, was used along with Newton's basic laws of motion to derive average speeds on grades (3). The results for two-lane rural highways are shown in Figure 3. Figure 2 and Figure 3 can be used to determine the PCEs of recreational vehicles at various speeds.

Table 1 gives PCEs of recreational vehicles, trucks, and buses on two-lane highways on specific individual subsections or grades. Table 2 gives average generalized PCEs of trucks, buses, and recreational vehicles on two-lane highways over extended section lengths. The following rationale was used in preparing Table 2:

1. The average speed of trucks, buses, and recreational vehicles (and even to an extent passenger vehicles) decreases as they move from level terrain to rolling and then to mountainous terrain;
2. The average speed of all types of vehicles de-

creases as they move from level of service A to level of service E (capacity); and

3. The speeds were selected for various vehicles for level of service A for the different terrains based on spot speed studies and on experience and judgment regarding the performance of trucks, buses, and recreational vehicles relative to each other.

PCEs may be determined for passenger cars because field studies demonstrated that passenger cars are also susceptible to speed reductions on steep grades. However, the use of PCEs in highway capacity computations is not included because, by accepted definitions of levels of service, which incorporate speed, the values are meaningless.

The procedure given in the HCM for computing the service volume for a two-lane highway is

$$SV = 2000\,(V/C)\,W_L T_C B_C \tag{5}$$

where

SV = service volume,
V/C = volume to capacity ratio,
W_L = adjustment factor for lane width and lateral clearance at a given level of service,
T_c = truck adjustment factor, and
B_c = bus adjustment factor.

Equation 5 may be modified to account for the effect of recreational vehicles as follows:

$$SV = 2000\,(V/C)\,W_L T_C B_C R_C \tag{6}$$

where R_c = recreational vehicle adjustment factor.

Equation 6, however, can introduce errors in capacity and service volume calculations. Rather than consider the adjustment factors separately to convert from the base volume into a mixed volume, equation 7 suggests a procedure that considers trucks, buses, and recreational vehicles in combination instead of separately.

$$C_c = 100/(100 - P_t - P_b - P_r + P_t E_t + P_b E_b + P_r E_r) \tag{7}$$

where

C_c = combined adjustment factor,
P_t = percentage of trucks
P_b = percentage of buses,
P_r = percentage of recreational vehicles,
E_t, E_b, and E_r = PCEs for trucks, buses, and recreational vehicles respectively.

The importance of combining the adjustment factors as shown in equation 7 was tested, and the percentage of error introduced in capacity and service volume calculations by considering the factors separately could range as high as 10 to 20 percent for mountainous terrain. In general when the adjustment factors are considered separately the following errors result:

1. As the PCEs increase, the resulting error increases;
2. As the percentage of vehicles other than passenger cars increases, the resulting error increases; and
3. As the number of variables increases, the resulting error increases.

Conditions on the Canadian transmountain two-lane highway system cause high PCEs. These conditions include steep grades and a high percentage of vehicles other than passenger cars. Thus, equation 6 may be written as follows:

$$SV = 2000\,(V/C)\,W_L C_c \tag{8}$$

SOME BASIC PROBLEMS IN DETERMINATION OF EQUIVALENCIES AND USE OF HIGHWAY CAPACITY EQUATION

In performing highway capacity (service volume) calculations over extended section lengths, one should correctly classify the highway section under consideration. Table 3 provides a guideline to correct classification. However, in many cases classifying the site according to terrain still remains difficult. Therefore to test the sensitivity of the effect of incorrect road classification, and also the manner in which recreational vehicles can be handled, computations were performed for a section of two-lane highway with the following characteristics:

Speed limit, 100 km/h (60 mph)
Travel lanes, 3.75 m (12 ft)
Shoulders, 3.0 m (10 ft)
Passing sight distance, 80 percent
Traffic composition—trucks 2 percent, buses 1 percent, recreational vehicles 19 percent

The results of the calculations are given in Table 4. A wide range of values results when recreational vehicles are considered as passenger cars, trucks, buses, or recreational vehicles. The table also illustrates the values for various terrains.

SENSITIVITY ANALYSIS

PCEs for trucks, buses, or recreational vehicles at the slower speeds, i.e., less than 50 km/h (30 mph), increase rapidly as speed decreases; the PCE varies greatly for a small change in speed. Sensitivity testing was conducted on the simulation model developed for recreational vehicles; the results are given in Table 5. In summary, power, mass, rear axle ratio, and altitude are highly sensitive variables in the model. Frontal area and coefficient of drag are not so sensitive.

Because the variables mass and power cause large changes in speed, the testing was continued to include highway capacity calculations. For the 9 percent grade under consideration with 80 percent passing-sight distance, traffic composition was assumed to be 80 percent passenger cars and 20 percent recreational vehicles with no buses or trucks. The results are given in Table 6. The two variables tested (power and mass) were reduced 10 percent, and the capacity at level of service E was changed from 1080 to 560 vehicles/h and 1080 to 1780 vehicles/h—changes of 51.8 percent and 64.8 percent respectively. These values appear very unrealistic, particularly the capacity value of 560 vehicles/h for the power variable at a speed of 39.3 km/h (24.4 mph), which is near the speed of a roadway running at or near capacity. Such values cause the PCE-speed curves to be suspect and suggest that the equivalencies are obviously too high at the slower speeds.

To further substantiate this hypothesis, another approach was taken. The maximum capacity of 2000 passenger cars/h/lane was arrived at by observing the minimum headways of passenger cars at various speeds (Figure 4, 1) and then applying the relationship.

$$\text{Volume} = \text{speed}/\text{spacing} \tag{9}$$

A familiar bell-shaped volume-speed curve (Figure 4) results from the above relationship.

In the headway study of recreational vehicles, the PCE for recreational vehicles was approximately 1.6

for 1000-vehicles/h flow for the lane studied. Assuming that the space occupied by a recreational vehicle is 1.6 times that of a passenger car, a volume-speed curve for recreational vehicles can be derived by applying equation 8 (Figure 5). Because the new curve depicts 100 percent recreational vehicles, PCEs can be derived by applying equations 7 and 8. The resulting PCE-speed curve at capacity is shown in Figure 6.

The location of the new PCE-speed curve exhibits a drastic shift to the left with a great discrepancy in values

Figure 4. Distance headway-speed relationship for passenger cars.

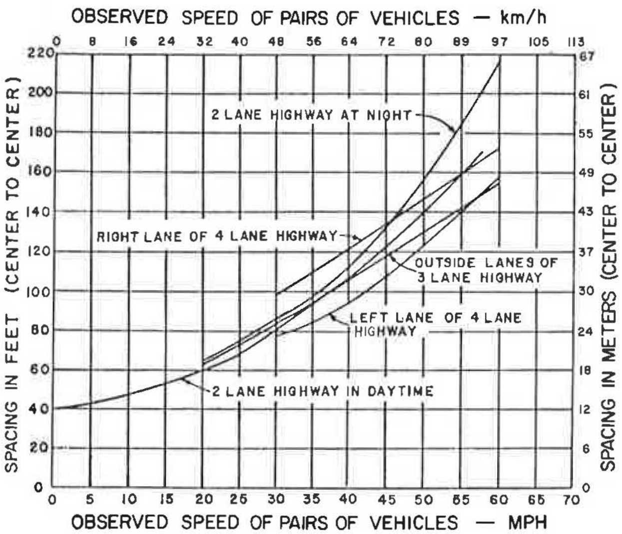


Figure 5. Daytime passenger car-recreational vehicle speed-volume relationship based on minimum headway spacing for two-lane highway.

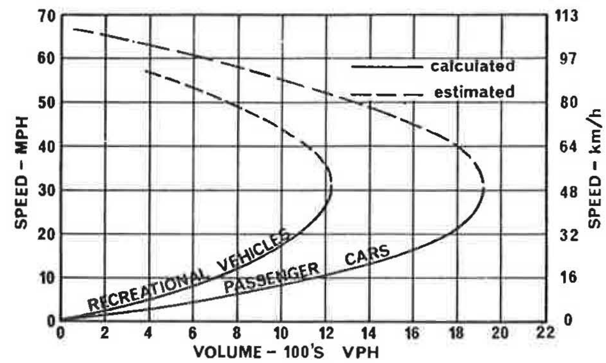


Table 4. Capacity of two-lane highway based on terrain, level of service, and recreational vehicle equivalent.

Terrain	Level of Service B	C	D	E	Recreation Vehicle Equivalent
Level	677	1180	1579	1949	Passenger car
	531	925	1421	1754	Recreational vehicle
	572	997	1421	1754	Bus
	555	967	1328	1639	Truck
Rolling	634	1105	1474	1820	Passenger car
	423	737	1055	1302	Recreational vehicle
	449	782	1110	1370	Bus
	376	655	871	1076	Truck
Mountainous	556	968	1271	1564	Passenger car
	345	602	781	965	Recreational vehicle
	321	560	698	862	Bus
	238	415	481	594	Truck

Note: All calculations are based on procedure used for combined adjustment factor (C_c) and PCEs given in Table 2.

Table 5. Sensitivity of variables in simulation model for vehicle performance on 9 percent grade.

Variable				Absolute Speed After 40 s (m/s)			Absolute Speed At 9.66 km (m/s)			Average Speed (m/s)			Average Speed Over 9.66 km (m/s)		
Name	From	To	Change (%)	From	To	Change (%)	From	To	Change (%)	From	To	Change (%)	From	To	Change (%)
Time, s	1.00	0.75	25.0	10.1002	10.0977	0.03	9.3130	9.3130	0.00	12.8246	12.8413	0.13	9.4684	9.4601	0.09
	1.00	1.25	25.0	10.1002	10.0446	0.55	9.3130	9.3130	0.00	12.8246	12.7900	0.27	9.4684	9.4659	0.03
	1.00	2.00	100.0	10.1002	9.9255	1.73	9.3130	9.3130	0.00	12.8246	12.6860	1.08	9.4684	9.4588	0.10
	1.00	3.00	300.0	10.1002	9.9710	1.28	9.3130	9.3130	0.00	12.8246	12.6934	1.02	9.4684	9.4595	0.09
	1.00	5.00	500.0	10.1002	9.9500	1.49	9.3130	9.3130	0.00	12.8246	12.6307	1.51	9.4684	9.4558	0.13
	1.00	8.00	800.0	10.1002	9.3562	7.37	9.3130	9.3130	0.00	12.8246	12.1189	5.50	9.4684	9.4226	0.48
Drag coefficient	0.60	0.65	8.33	10.1002	9.9055	1.92	9.3130	9.1288	1.98	12.8246	12.7011	0.96	9.4684	9.2832	1.96
Rear axle ratio	3.00	3.30	10.00	10.1002	10.9564	8.48	9.3130	10.8712	16.73	12.8246	13.2170	3.06	9.4684	10.9799	15.96
Frontal area, m^2	4.18	4.65	11.11	10.1002	9.9418	1.57	9.3130	0.0708	2.60	12.8246	12.7330	0.71	9.4684	9.2298	2.52
Power, kW	187.69	168.90	10.00	10.1002	8.8351	12.53	9.3130	7.2517	22.13	12.8246	12.2129	4.77	9.4684	7.4346	21.48
Mass, kg	3725.92	3316.59	10.00	10.1002	11.1208	10.10	9.3130	11.0869	19.05	12.8246	13.3084	3.77	9.4684	11.1908	18.19
Altitude, m	1890	1372	27.41	10.1002	10.9974	8.88	9.3130	10.8904	16.94	12.8246	13.2622	3.41	9.4684	11.0011	16.19
		Sea level[a]		10.1002	12.1281	20.07	9.3130	12.2544	31.58	12.8246	13.9886	9.08	9.4684	12.4990	32.00

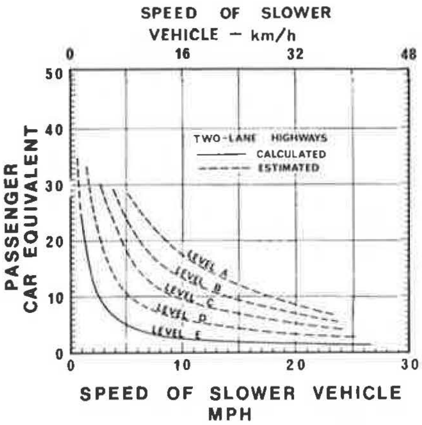
Note: 1 m/s = 3.281 ft/s; 1 kg = 2.205 lb; 1 kW = 1341 hp; 1 m = 3.281 ft; 1 m^2 = 10.764 ft^2.
[a]At sea level, vehicle was subject to cyclic gear changes; therefore, absolute speeds are not too meaningful.

Table 6. Effect of varying power and mass of recreational vehicles on passenger car equivalents and service volumes on 9 percent grade using 80 percent passenger cars and 20 percent recreational vehicles.

	Power			Mass		
Item	From 187.7 kW	To 168.9 kW	-10.0% Change	From 3725.9 kg	To 3316.6 kg	-10.0% Change
Speed, km/h						
Avg over 10 km	50.1	39.3	-21.5	50.1	59.1	+18.2
Levels of service B and C	6.8	12.0	+76.5	6.8	4.0	-41.2
Levels of service D and E	5.2	15.3	+194.2	5.2	1.6	-225.0
Vehicles per hour						
Service level B	322.0	217.0	-32.6	322.0	434.0	+34.8
Service level C	561.00	378.0	-31.7	561.0	756.0	+34.8
Service level D	875.00	454.0	-51.9	875.0	1441.0	+65.8
Service level E	1080.00	560.0	-51.8	1080.0	1780.0	+64.8

Note: 1 km = 0.6 mile; 1 kg = 2.20 lb; 1 kW = 1.341 hp.

Figure 6. Estimated placement of passenger car equivalent speed curves for two-lane highways.



at very slow speeds and, to a lesser extent, at higher speeds. Similar curves may exist for other levels of service as shown by the dotted lines in Figure 6.

FUTURE WORK

During our investigation several questions became evident; we present them here in the form of suggestions for further research.

1. A more precise definition of terrain is required to correctly classify extended highway subsections. Perhaps the British terms "hilliness" and "bendiness" might be applied to North America. Hilliness is defined as the total rise and fall per unit of distance, and bendiness, the total change of direction per unit of distance.
2. The present concept of level-of-service measures is primarily based on the idea that speed should be reevaluated. Reevaluation is necessary because many agencies are adopting the 88-km/h (55-mph) speed limit and the volume/capacity ratios for the different speeds are felt to be no longer valid. By the speed definition level of service A no longer exists.
3. The methods for determining PCEs of slower vehicles need to be reviewed. The validity of the PCE for vehicles at very slow speeds is doubtful.

CONCLUSIONS

The research project was able to uncover the underlying methods for generating PCEs and replicate the PCE-speed curves contained in the HCM. By applying the same principles with a vehicle simulation model, adjustment factors for recreational vehicles were derived. However, subsequent work in applying the new adjustment factors to a field site and in carrying out sensitivity testing in highway capacity computations produced strong evidence that the new adjustment factors and those for trucks and buses contained in the HCM may not be valid and may require further refinement. There is a need for further research for the determination of PCEs of larger and slower moving vehicles for two-lane rural highways.

ACKNOWLEDGMENTS

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Discussion

Arthur A. Carter, Office of Traffic Operations, Federal Highway Administration

The authors should be commended for undertaking this research. The increasing presence of recreational vehicles in the traffic stream in Canada is, of course, equally evident in the United States, and the findings, therefore, are equally needed here. I am familiar with the work, having corresponded with Werner on several occasions regarding it and having reviewed the complete report on which this paper is based.

Two particular factors distinguish the recreational vehicle problem from the overall problem of slow-moving vehicles in the traffic stream. First, the largest concentrations of recreational vehicles are likely to be found on those very highways least appropriate for expansion to multilane design because of environmental considerations—highways in or near parks or other scenic attractions. Second, some recreational vehicle drivers are relatively unfamiliar with their vehicles, as compared to typical truck and bus drivers, and drive hesitantly, erratically, and sometimes carelessly. The problem, then, is a real one not subject to easy correction. It needs to be included in highway capacity criteria. The authors' contribution to development of an understanding of it is therefore welcome. The fact that their work was based on detailed studies of actual traffic flows in western Canada lends credibility to the findings.

Particularly welcome is their reporting of results in a format directly supplementing the tables in the 1965 HCM. Anyone familiar with HCM procedures can immediately apply the new findings.

In this connection, the authors express concern regarding the "error" presumably introduced by using three separate adjustment factors for trucks, buses, and recreational vehicles individually as multipliers, as would be done if the HCM procedures are followed. They suggest substitution of an overall combined adjustment factor, particularly where many vehicles other than passenger cars are in the traffic stream, and equivalencies are large as in mountainous terrain. This discrepancy between the two methods was recognized in the past, but rarely was it a problem. Seldom were bus volumes large enough to warrant an adjustment separate from that for trucks. At the time that the current procedures were completed, simplicity of use of separate multipliers, given the limited precision of the overall method, overshadowed any potential refinement of a combined factor. Now, with allowance necessary for large volumes of recreational vehicles as well as significant volumes of tour buses on the same recreational routes, the authors are fully justified in again suggesting a refined procedure.

The authors refer to the Walker method for deriving vehicle equivalencies as having been the basis for much of their work on performance on grades. The method, which appears somewhat complex as summarized briefly here, is the procedure that was used in development of equivalencies reported by Schwender, Norman, and Granum (6). The detailed procedures were never formally published, but W. P. Walker of the then Bureau of Public Roads retained them in his files. In the mid-1960s they were applied to develop the equivalencies for all levels of service on two-lane, two-way highways, which appear in chapters 5 and 10 of the 1965 HCM, and several researchers have recently applied them in Europe and elsewhere.

Questions have since been raised regarding the validity of the resulting equivalencies and about the method itself. In particular, the logic of the relationships between comparable values at the several levels of service has been debated, since the values contradict in some respects the original values appearing in the Schwender, Norman, and Granum paper (6), which represented three levels of operation at and near current level of service B. Further questions could be raised regarding the present-day validity of the data to which the method was applied in the 1950s and the 1960s. But the authors appear to have found the basic concept of the methods acceptable. Further, their Figure 2 generally confirms the relationships between levels of service as shown in Figure 5.6 in chapter 5 of the HCM, if not the absolute values. By using the concept, they have produced results consistent with the HCM.

Interestingly the curves for lower speeds shown in Figure 5, which the authors derived partially on speculation by using headway concepts and equations developed by A. D. St. John of Midwest Research Institute, have relationships contradicting Figure 2. That is, the order of the family of curves is reversed. The logic of this reversal is questionable. The figure did not appear in earlier work on which this summary paper was based, but that work did include discussion of the differing equivalencies obtained by the "passing" versus the "headway" methods. The larger of the two equivalencies was preferred, but the reversal problem was not mentioned.

The authors have gradually changed their viewpoint, as they have further analyzed their work, and now see increasing validity in the headway-derived values, which produce lower equivalencies. This whole topic is open to question. Historically, the headway approach has proved subject to pitfalls. The widely varying results shown in Figure 1 of the original 1950 HCM graphically illustrate this point. In particular, wide errors have occurred in predicting overall hourly traffic performance from the performance of one-directional platoons of vehicles on two-lane, two-way highways.

Capacities and flow rates on such highways must be quoted as totals for both directions because of the shared use of lanes for basic flows and opposing-direction passing. "By-lane" flow rates have proved to be misleading and usually excessive, unless carefully interpreted. (St. John of Midwest Research Institute is contending with this problem and has recently developed some interesting views.)

The point that the authors raise regarding passenger car performance deterioration on long steep grades is well taken. Policy of the American Association of State Highway Officials indicates that most passenger cars can negotiate grades up to 7 or 8 percent without appreciable speed loss. The HCM is slightly more conservative and states that capacity is seldom affected by passenger car performance on such grades; that is, they can almost always maintain the 48.27 to 56.32-km/h (30 to 35-mph) speeds at which capacity occurs. The latter appears to be more likely today; the better levels of service, requiring higher speeds, are being increasingly influenced by the reduced performance found in many modern cars.

In a related matter, the authors recommend that the entire level of service-speed scale be reevaluated in the light of the national 88.5-km/h (55-mph) speed limit. This would seem to be premature. Admittedly, level A performance is not now permitted, but the scales are based on driver desire, not performance. Until and unless drivers come to accept 88 km/h (55 mph) more fully than they do now, labeling this speed as level A seems incorrect.

In summary, the authors have provided a much-needed, practical, immediately useful addition to the state of the art in their expansion of the Walker, or passing, method to cover recreational vehicles. Their findings with respect to the headway method appear more speculative.

I hope that this research can be used with other recent relevant studies (particularly those under the National Cooperative Highway Research Program by St. John, who, incidentally, used the authors' field data) for eventual development of updated equivalency procedures suitable for replacement of the current HCM procedures.

Authors' Closure

We wish to comment briefly on some of the important points stressed by Carter. We agree that our viewpoint has changed during the course of our recreational vehicle project and now see increasing validity in the headway-derived values. However, as correctly stated by Carter, the entire topic is open to question. Our recommendation that the entire level of service-speed scale be reevaluated still stands. In the mountainous regions of British Columbia, where the speed limit is 88.5 km/h (55 mph), two-lane highways wind along cliffs, and a high percentage of trips are highly recreational, drivers may not experience level of service A performance as defined in the HCM. However, under these circumstances this speed may well be their desire. We agree that the next stage of research should be in conjunction with the recent work of St. John.

Abridgment

Study of Location Bias in Speed-Volume Relationships for Two-Lane Arterial Roadways

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Speed, volume, and density are the fundamental characteristics of traffic flow on roadways (1, 2). These relationships are not widely applied in practice, however, for several reasons. One reason is the question of the influence of location bias on the speed-volume-density relations determined for a roadway. In theory, these relationships are valid only at the location at which they are determined. This fact implies that the speed-volume-density functions would have to be determined for each and every point in the road system. The purpose of this study was to investigate the location bias in speed-volume relationships for a sample of similar locations on two-lane, two-way arterial roadways.

Study locations were selected on the basis of similarities in traffic, abutting land use, geographic location, roadway features, and traffic control. Five locations in Macomb County, Michigan, a suburban area in the Detroit metropolitan region, were selected on the basis of the above considerations. These locations have a relatively straight horizontal alignment with no appreciable grade or grade changes. The basic land use consists of single-family residential developments along one side of the roadway, vacant or sparse development on the opposite side, and isolated commercial development near major intersections. All locations had a posted speed of 72.4 km/h (45 mph). The traffic volumes and traffic composition were about the same for each location. All of the locations were situated at points in the same general geographic area on a 1.6-km (1-mile) grid arterial network having traffic signals at the intersections. The study locations chosen were, however, at varying distances (0.5 to 1.1 km or 0.3 to 0.7 mile) from the signalized intersections. All sites had either 3.04 or 3.34-m (10 or 11-ft) lanes.

Spot speed and volume data were collected at each location for each direction of traffic flow, thus giving a sample of 10 sites. The spot speed data were reduced to space mean speed by taking the harmonic mean of the spot speed collected by a radar meter. The volume counts were taken during the speed checks and were expanded to hourly flow rates. A corresponding traffic density was calculated as the quotient of the flow divided by the space mean speed for each data point.

The speed and volume data points were plotted, and curves were hand-fitted through the points for each study site. The speed-volume plots for each site were compared on a composite plot. These comparisons focused on the volume range of 0 to 1200 vehicles/h/lane since only a few data points were observed above this upper limit. The composite plot revealed that, up to 800 vehicles/h, all of the speed-volume curves did lie in a band having a width of about 4.8 km/h (3 mph). Beyond 800 vehicles/h the curves became more diverse; a maximum variation of about 12.8 km/h (8 mph) occurred at 1200 vehicles/h. In this higher volume range, the curves were noted to cluster into two groups each having a band of about 4.8 km/h (3 mph) in width. Labeling the individual curves showed that the higher cluster on the speed axis included all of the study locations lying opposite the residential land use. The lower cluster contained all of the locations adjacent to the residential development. Further analysis indicated the existence of an influence of lane width on the relative location of the specific curves in a cluster. Sites having the 3.04-m (10-ft) lanes generally had a lower speed-volume curve than the other sites. The distance of the study from an upstream or downstream signal did not appear to have an influence, since the locations were far enough away from the signalized intersections.

CONCLUSIONS

1. For volumes up to 800 vehicles/h on two-lane arterials, the speed-volume relationships of similar roadway locations lie within a narrow speed band having a width of about 4.8 km/h (3 mph).
2. Beyond 800 vehicles/h the speed-volume curves are influenced by the type of abutting land use. The more intense the land use is, the lower the speed-volume relationship is.
3. The variations in speed-volume curves attributable to location bias are small, given similar two-lane arterials

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and abutting land uses. Hence, location bias can be ignored for practical application in the design of traffic control systems.

Further research is necessary to investigate the location bias characteristics of other types of roadways and other typical roadway environments and to define the average speed-volume-density relationships and their associated confidence limits.

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Capacity Evaluation of Two-Lane, Two-Way Highways by Simulation Modeling

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A tool developed during the last four years, a microscopic Monte Carlo simulation model of a two-lane, two-way highway, was used to evaluate capacity more accurately. The model operates by processing individual vehicles traveling along a two-lane road where grades and no-passing zones can be specified. The performance of passenger vehicles and trucks is modeled in detail. The validity of the model is demonstrated by comparing specific simulated output to published data obtained under similar conditions. The model is applied in a comparison with the analysis procedures given in the Highway Capacity Manual for two-lane, two-way roads. Satisfactory agreement is obtained between the manual operating speed-volume to capacity ratio curve and a similar relation obtained from model runs. Poorer agreement is obtained between the manual truck equivalency factors for two-lane, two-way roads and similar factors derived from model runs. The conclusion is that the manual may overestimate the adverse effects of trucks on steeper grades. This paper should be of interest to practicing engineers because it introduces an important new tool for detailed evaluation of traffic operations on two-lane highways, and it provides evidence that revision is needed in two-lane highway analysis procedure in the Highway Capacity Manual.

Chapter 10 of the 1965 Highway Capacity Manual (HCM) (1) presents methods for determining the effectiveness of operations on existing or proposed two-lane, two-way highways. This methodology, used for over 10 years, in many instances gives satisfactory, though conservative, results. Also the methodology is somewhat vague, and unsuitable for analyzing complex geometrics such as compound grades with irregularly intermixed passing and no-passing zones. Further, little information is provided regarding the microscopic aspects of the traffic flow. The methodology is based largely on the West Virginia study (2), dating from the 1950s, and so particular parameter values are proving to be out of date. Especially critical are the truck equivalency factors used by the HCM (1), based on performance tests of a single truck model conducted in the West Virginia study (2).

A recently developed Monte Carlo simulation model (3) of a two-lane road (SIMTOL) is used to assess questionable areas of previous studies. This model functions both as an aid in assessing generalized methodologies, as in chapter 10 of the HCM, and as a tool for the direct prediction of the operational effectiveness of roads with traffic characteristics or geometry too complex to be accurately assessed by using generalized techniques.

This paper presents the simulation model, validation results, and application in comparison with the HCM procedures for the analysis of two-lane highways. Particularly important are the conclusions regarding the need for improvement in the truck equivalency factors for two-lane roads presented in the manual.

THE SIMTOL MODEL

The SIMTOL model considers cars and trucks. Cars are any vehicle with a low mass-to-power ratio, roughly 36.5 kg/W (60 lb/hp) or less. This includes standard passenger cars and most pickup trucks and vans. Recreational vehicles and cars heavily loaded for recreational uses (motor homes, campers, car-trailer combinations) are beyond the scope of the model. Trucks are commercial vehicles with high mass-to-power ratios and are subdivided into six classes as shown in Figure 5.3 of the HCM (1). Because of the lack of calibration data on trucks, the following report assumes that trucks do not pass. This limitation requires future research. Further, this report refers to both cars and trucks collectively as vehicles.

A number of additional assumptions have been made to limit the scope of the model to a tractable system. Only good-weather, daytime driving conditions are considered. Also, realistic applications of the model are limited to two-lane highways of fairly high design standards since speed reductions on horizontal curves are not modeled. Nor are speed reductions for comfort or caution on vertical curves modeled. Only one direction of traffic is explicitly simulated; in the other direction all simulated vehicles move at a constant speed, but with random headways. These two directions of traffic are referred to respectively as the primary direction and the opposing direction. This last assumption is not as surprising as it might seem. The two directions of traffic interact only through passing maneuvers. Studies

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of passing behavior by the Franklin Institute Research Laboratories (FIRL) (4) show that drivers considering a passing maneuver are primarily sensitive to the mean speed of opposing traffic, probably because these drivers cannot perceive the speed of an oncoming vehicle until it is quite close. In addition, considerable care has been taken to ensure that the user has the option of specifying a reasonably realistic headway distribution for the opposing lane. Project time, budget limitations, and computer efficiency considerations precluded the elimination of the above simplifying assumptions; such improvements are left for future research.

The model is based and calibrated to the extent possible on validated models of other authors (incorporated as submodels). However, in a few instances in which information is lacking, specific assumptions are necessary regarding the form of parameters for certain vital relationships. Budget limitations precluded a new data collection program within this project; the assumed relations appear adequate and have been held to a minimum.

In spite of the limitations described above the capabilities of the model are extensive. The path of each simulated vehicle is traced as it travels along the road. Each driver attempts to maintain a desired speed, but may be prevented from doing so by other drivers or by the limitations of the vehicle. Simulated cars, trapped behind slower vehicles, attempt passing maneuvers. These maneuvers may be successful or may be aborted. Decelerations of trucks on upgrades are modeled in detail by using an analytical submodel. Vehicle performance parameters and driver behavioral characteristics (such as desired speeds) are all quantified as appropriately distributed random variables.

An arbitrarily complex road geometry that satisfies the above assumptions can be specified by a user. For example, no-passing zones can be alternated with passing zones to reflect sight distance restrictions. Less restrictive sight distance limitations within passing zones can also be indicated. Up and down grades and compound grades can be specified. An arbitrary mix of cars and trucks can be specified; the trucks are further stratified into six classes. Thus, realistic configuration of present traffic on a present two-lane road can be simulated.

The entire section of road to be simulated is divided into adjacent but nonoverlapping units called subsections. Within these subsections all roadway characteristics are held constant. Sections of road with characteristics that change in some continuous manner (such as a vertical curve) can be approximated in the model by a series of adjacent, short subsections. Similarly, time is automatically divided by the model into fixed, short time increments of lengths (Δt). The size of these increments, typically 1 or 2 s, is selected by the user.

Simulated vehicles in both primary and opposing directions arrive at their respective entries to the road section with random headways. A probability theory (5, 6) may be applied to the distribution of these headways, or the headways can be exponentially distributed. A warm-up period and an optional warm-up level subsection for primary direction vehicles are included to eliminate transient effects.

Structure

The model is organized through operating modes (3) as shown in Figure 1. During every time interval (Δt), every simulated vehicle is assumed to be traveling in one and only one operating mode. The model logic is constructed so that one mode never overlaps another. The following seven operating modes are defined in the model and collectively describe all possible operating conditions on the simulated road.

1. Desired speed is selected when the driver is unimpeded by his vehicle's performance or by other vehicles. Desired speeds follow separate normal distributions; parameters are specified by the user.
2. Car following is used when a driver decelerates to match speeds with a lead vehicle. Car following employs a log-normal distribution of car-following distance whose mean is a function of speed (7); the distribution is modified to allow for tailgating (3).
3. Performance limited is used primarily for drivers of simulated trucks unable to maintain desired speeds on or following upgrades. Performance characteristics of an individual simulated truck are determined by its mass-to-power ratio. This parameter is assumed to be distributed among the U.S. trucks as specified by the curves in Figure 3 of Wright and Tignor (9), reproduced as Figure 5.3 of the HCM. These curves show the distribution of mass-power ratios within various axle configurations; however, the specification of the percentages of trucks within these classes is specified by the SIMTOL user. The performance of each individual truck is simulated in detail in SIMTOL, using the model of Firey and Peterson (10).
4. Acceleration after pass enables the driver to attain the desired speed after completing a pass. Acceleration employed in this mode is a function of the acceleration employed during passing.
5. Deceleration after pass enables the driver to decelerate after completing a pass (coasting in gear with foot off the accelerator and without braking is the assumed behavior). Decelerations are quantified from data in Table 2.5 of the Traffic Engineering Handbook (11).
6. Flying pass and accelerative pass, modes 6 and 7 respectively, both use acceleration and depend on the randomized individual performance capabilities of the passing car. A flying pass is initiated at a speed greater than, or equal to, the passing driver's desired speed; an accelerative pass is initiated at a speed less than the driver's desired speed. Acceptance or rejection of each passing opportunity is determined by using gap acceptance curves developed from U.S. studies by FIRL (4) and Swedish studies by Ahman (12).

In commencing the simulation the time periods can be thought of as starting at time t_b, being incremented in units of Δt, and ending when t_e is attained (i.e., $t = t_b, t_b + \Delta t, \ldots, t_e - 2\Delta t, t_e - \Delta t, t_e$). At each time scan point t the position of every vehicle on the road is updated. First, opposing vehicle positions are updated; then primary direction vehicle positions are updated, a more complex task because speeds and modes are included. Primary direction vehicles are processed from downstream to upstream, so that the actions of following vehicles can be simulated based on the actions of lead vehicles. The actions of every primary direction vehicle are analyzed at every time scan point t by the logic of the mode-to-mode matrix shown in Figure 1. Given the mode that resulted at the vehicle's previous time scan point $t - \Delta t$, only specified new modes are possible. These new possibilities are given by the matrix. Each cell containing an asterisk designates a possible mode-to-mode transition; blank cells represent impossible transitions. This routine must be recalled if the vehicle being processed has entered a new subsection at some time during the interval ($t - \Delta t$, t), to take account of the effects of possibly different subsection geometric characteristics. After every vehicle has been processed, the exit point of the road is checked for the removal of vehicles from the system. If needed, new vehicle arrivals are generated and entered down the road. The entire process is repeated until the simulation clock reaches t_e.

Computerization

The model has been programmed in FORTRAN IV for a digital computer and has been run on a CDC 6400. Computer running time is variable, depending on many parameters, especially the expected number of vehicles to be simulated. An example run for 4.8 km (3 miles) of road, including a 1.61-km (1-mile) warm-up subsection and a grade, took 30 s of computer time to amass a 200-vehicle sample. The simple program form consists of three sequential blocks: input-initialization, simulation, and output. The input for a single simulation run consists of only a few (usually 10 to 20) cards. The required input data can be divided into three categories: traffic parameters, roadway parameters, and run parameters. The traffic parameters specify arrival distributions, expected flow for the primary and opposing directions, expected percentage of trucks and distribution of trucks within the subclasses, and desired speed distributions. The roadway parameters primarily specify length, steepness of grades, passing and no-passing zones, and sight distances. The run parameters specify timing and accounting details such as period of time to be simulated, time increment Δt, random number seed, and various output control options. The basic model output is a prediction of the highway performance in terms of such variables as travel times and travel speeds, spot speeds and time headways at specified observation stations along the road, and platooning. An optional binary tape containing detailed simulated microscopic data can be requested. In addition, histograms can be drawn on the printer showing the observed density functions of most output random variables (Figure 2), and a time-space diagram graphically showing individual vehicle trajectories can be drawn. (Since this model was designed for customary units only, values in this and other figures are not given in SI units.)

Figure 1. SIMTOL mode-to-mode matrix.

From Mode \ To Mode		Name:	Desired Speed	Car Following	Performance Ltd.	Acceleration after pass	Deceleration after pass	Flying pass	Accelerating pass
		No.	1	2	3	4	5	6	7
Name	No.								
Desired Speed	1		*	*	*a			*b	
Car Following	2		*c	*	*				*
Performance, Ltd.	3		*	*	*				
Acceleration after pass	4		*	*	*d	*			*b
Deceleration after pass	5		*	*	*a		*	*	
Flying pass	6			*	*e		*	*b	
Accelerating pass	7		*	*	*e	*	*	*	*

Dashed lines denote submatrices used in processing. * in cell indicates a possible transition; blank cells represent impossible transitions.

Notes:

a. Can only occur at the time a new subsection is entered, since grade changes only occur at subsection boundaries.

b. Can only occur when the transition from Modes 1, 4, or 5 to Mode 2 will occur if the passing opportunity is not accepted.

c. Can occur if lead vehicles accelerates to a speed greater than the follower's desired speed.

d. Can occur at the time a new subsection is entered, and rarely, when passed vehicle's speed < passer's crawl speed < passer's desired speed.

e. Can occur only at the entry to a new subsection and causes the pass to be immediately aborted.

f. Very rare; probability for such a transition virtually zero.

VALIDATION OF THE MODEL

Component validation assessed the realism of separate subparts; system validation assessed the realism of the subparts working in coordination with one another.

Component Validation

Component validation, done during formative stages of model development, resulted in early changes for realism. For example, during early development of the passing submodel, simulated passing times were excessive compared to FIRL data (4). Therefore pre-pass tailgating was incorporated into the car-following submodel, and acceptable simulated passing times were achieved (3).

One of the more important component validations performed on SIMTOL was of the truck performance submodel, Firey and Peterson's model (10) with randomly distributed mass-power ratios from Wright and Tignor (9). The comparative actual human behavior data were obtained from the published results of a study by Williston (13). Williston used a radar meter to sample the speeds of trucks unimpeded by other traffic on four-lane Connecticut highways. Although the highway used in SIMTOL is two-lane, comparison was felt to be acceptable because only unimpeded trucks, unaffected by their surrounding traffic environment, were considered. The comparative SIMTOL runs simulated only unimpeded trucks. Figure 3 shows the comparison of the simulated and observed data for tractor-trailer trucks at Williston's study site 3 (13). The agreement is very close. For the faster single-unit trucks, the modeled speeds are somewhat higher, to about 13 km/h (8 mph), than the corresponding speeds observed by Williston. However, this discrepancy seems to be largely attributable to one or two sources of bias caused by Williston's data collection technique. Sampling only unimpeded trucks introduces a negative or downward bias into the sample because lighter (faster) trucks, which are more likely to be impeded by other traffic, are underrepresented. A negative bias could also occur if the radar meter were obvious to the passing drivers, again probably affecting the faster trucks most. In both cases, the biases would cause the speeds of the faster trucks to be depressed, precisely the behavior observed. Similar comparison runs were also made for Williston's other study sites, again with acceptable results (3). Therefore, the SIMTOL truck submodel has been accepted as performing in a realistic manner.

System Validation

System validation runs were made against five of the six speed distribution curves shown in Figure 3.28 of the HCM (1), which represent typical behavior at various flow levels for a two-lane, two-way highway under ideal conditions (level, no trucks, good sight distances). The manual does not specify whether these speed distributions were taken over space or time, so for convenience they were assumed to be taken over time, enabling their comparison to spot speeds output by SIMTOL. A normal approximation to the speed of 200 vehicles/h [mean 88.5 km/h (55 mph), standard deviation 14.5 km/h (9 mph)] was used as the desired speed distribution for all flow levels within the simulation. The comparative SIMTOL spot speed distributions were taken 6.4 km (4 miles) from the entry point of the simulated road to ensure the elimination of transient effects. Arrivals in both simulated directions were exponentially distributed; the expected traffic flows were split equally between the directions. Of course, the SIMTOL runs were made for the same ideal conditions. The results of these runs are shown in Figure 4. The agreement between the observed (presumably smoothed) and simulated spot speed distributions is very close. For all five simulation runs, the time mean speeds also all agree with the means of the observed curves to within a 3-km/h (2-mph) range. The simulated distributions evidence an increasingly discrete nature with increasing flow level because most vehicles are platooned for high flow levels and all vehicles within a platoon are traveling at nearly the same speed. Therefore, the 1700-vehicles/h curve is a sampling of only about eight platoons. The simulation sample sizes were based on the criterion that at least eight platoons be sampled, but that the sample size never be fewer than 100 vehicles. This criterion dictated expected sample sizes of 100 vehicles for flow levels of 200 to 1400 vehicles/h and 200 vehicles for flow levels of 1700 vehicles/h. The criterion was compulsory be-

Figure 2. Sample spot speed histogram output from SIMTOL.

```
SPOT SPEED MEASUREMENTS AT STATION 1,
   BETWEEN SUBSECTIONS  0 AND  1 AT  X = 0.0FT

   RANGE        NUM.
    MPH         OBS.
                        +++++++++++++++++++++++++
 41.-   42.      0  +
 42.-   43.      2  +**
 43.-   44.      2  +**
 44.-   45.      0  +
 45.-   46.      2  +**
 46.-   47.      1  +*
 47.-   48.      2  +**
 48.-   49.      2  +**
 49.-   50.      7  +*******
 50.-   51.      3  +***
 51.-   52.     10  +**********
 52.-   53.     11  +***********
 53.-   54.     11  +***********
 54.-   55.     17  +*****************
 55.-   56.      5  +*****
 56.-   57.     10  +**********
 57.-   58.      5  +*****
 58.-   59.      4  +****
 59.-   60.      3  +***
 60.-   61.      5  +*****
 61.-   62.      3  +***
 62.-   63.      2  +**
 63.-   64.      0  +
 64.-   65.      2  +**
 65.-   66.      1  +*
 66.-   67.      0  +
 67.-   68.      0  +
 68.-   69.      1  +*
 69.-   70.      0  +
 70.-   71.      1  +*
 71.-   72.      0  +
 72.-   73.      0  +
 73.-   74.      0  +
 74.-   75.      1  +*
 75.-   76.      0  +

MEAN SPEED          =     54.560       MPH
EST. STD. DEV.      =      5.282       MPH

  CARS              =    104.
  TRUCKS            =      9.
                         -----
  N (TOTAL)         =    113.
```

Figure 3. Observed versus simulated speeds for tractor-trailer trucks.

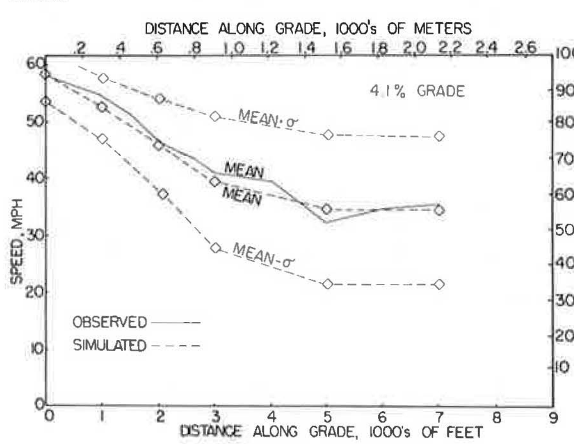


Figure 4. Simulated versus capacity manual speed distributions.

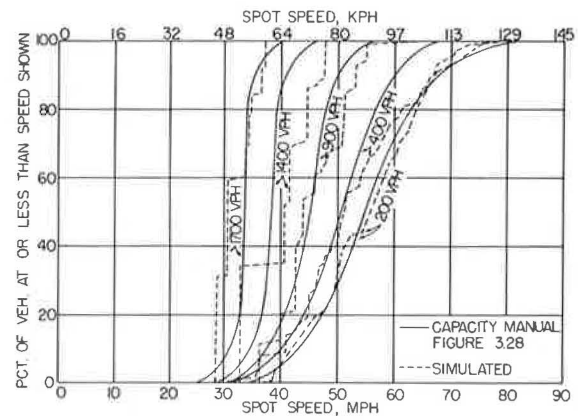


Figure 5. Simulated versus observed travel speed distributions on Ca-37.

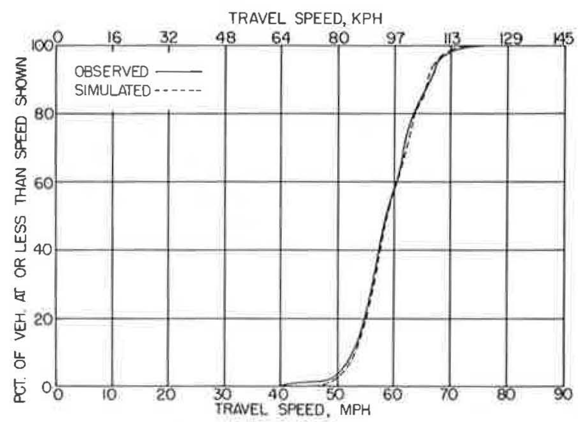
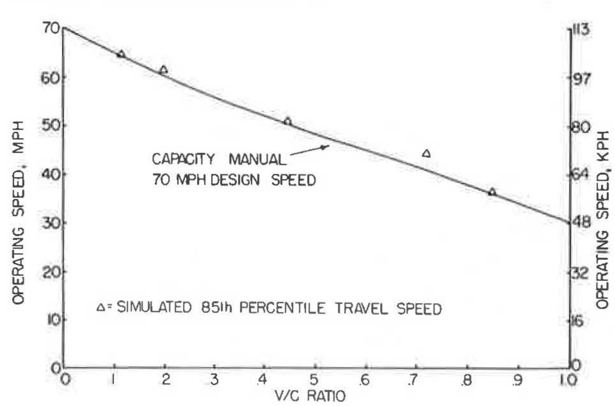


Figure 6. Operating speed versus v/c ratio.



cause the limited project budget kept the acceptable sample size to a minimum. Nonetheless, comparison to the HCM is still valid, especially in view of the overall similarity of results to the set of distributions in Figure 4 and the agreement in mean speeds.

Additional system validation runs were made for the above HCM curves by using Schuhl (5) headways for the opposing traffic. These results agreed nearly as well. Data were also obtained for a two-lane California highway (Ca-37) west of Vallejo. When comparative SIMTOL runs were made for the specific configuration of Ca-37, a satisfactory comparison was again obtained (3), as shown in Figure 5. As a result of all validations, SIMTOL is believed to yield realistic results and can indeed be a useful tool in assessing the operations of two-lane, two-way roads.

APPLICATION TO THE HCM

Next an evaluation was conducted of the accuracy of the methodology presented in the HCM for two-lane highways. Because budget limitations prevented simulation runs of large sample sizes, the following analyses do not yield accurate new quantitative values but emphasize obtaining approximate estimates of certain variables used in the manual. The overall behavior of the variables can then be compared with the HCM and areas for further research and possible revision pointed out.

The basic equation given by the HCM for the analysis of two-lane highways is

$$SV = 2000\,(v/c)\,W_L\,T_L \qquad (1)$$

where

- SV = service volume in mixed vehicles per hour,
- v/c = volume to capacity ratio,
- W_L = width adjustment factor, and
- T_L = truck adjustment factor.

Since SIMTOL is not specifically concerned with lane width (lane width is reflected only through the desired speed distributions input by the user), this analysis considers only situations of adequate width. Therefore, W_L has been set to 1.0, and, since SIMTOL assumes a modern alignment, all comparisons have been based on the 130-km/h (70-mph) design speed curves in Figure 10.2a of the HCM. Thus, only the v/c ratio and T_L terms are examined in the next two sections.

Basic Equation Under Ideal Conditions

This section considers equation 1 under ideal conditions, that is, with no trucks and passing sight distance everywhere in excess of 457 m (1500 ft). Therefore, the truck adjustment factor T_L is set to 1.0, and equation 1 simplifies to

$$SV = 2000\,(v/c) \qquad (2)$$

The limiting value of v/c for each level of service is given in Table 10.7 of the HCM as a function of a number of variables, mainly, operating speed—the primary measure of the level of service in the manual. The limits for the v/c ratio given in Table 10.7 were determined directly from the four-curve families of Figure 10.2 of the manual. The accuracy of these curves is therefore central to the methodology for the capacity analysis of two-lane highways.

SIMTOL operating speed might be considered as the average realized travel speed of that group of vehicles whose desired speed is within 3 to 5 km/h (2 to 3 mph) of the design speed. Unfortunately, SIMTOL does not use the design speed as an input; rather design speed enters the model only through the choice of the desired speed distribution. Instead, the 85th percentile overall travel speed obtained from the simulation is used as a substitute for operating speed. Although operating speed and 85th percentile travel speed are not quite the same, the comparison is acceptable because of the agreement of the 85th percentile speeds in Figure 3.28 of the HCM (reproduced in part in Figure 4) with the operating speed for a 130-km/h (70-mph) design speed shown in Figure 10.2a of the manual (Figure 6). The 85th percentile travel speeds obtained from the SIMTOL runs for ideal conditions used for Figure 4 are also portrayed in Figure 6, and since the correspondence between the curves in Figure 4 is close, the correspondence in Figure 6 is also close. Thus, the 85th percentile travel speed obtained from SIMTOL corresponds closely to the operating speed used in the manual; therefore the modeled results and the manual are in close agreement under ideal conditions.

Basic Equation Under Truck Grade Conditions

On upgrades trucks decelerate, adversely affecting other traffic. When this is quantified, a wide range of parameters must be considered, including grade steepness and length, percentage of trucks, mix of truck types, traffic flow, and split of the flow between the directions.

The HCM quantifies the effect of trucks by using the truck adjustment factor T_L identified in equation 1. However, W_L has already been assumed to be 1.0; therefore, equation 1 reduces to

$$SV = 2000\,(v/c)\,T_L \qquad (3)$$

The factor T_L is in turn based on the passenger car equivalent, E_T, for a truck and the percentage of trucks in the stream. E_T is the number of passenger cars one truck is supposed to equal. In the HCM, E_T is a variable dependent on steepness and length of grade and flow (i.e., level of service). E_T is specifically assumed to be independent of the percentage of trucks on the highway and is also generally assumed to be independent of the split of flow between directions. The values for this factor are based on the deceleration profiles of the one supposedly average 1950 model truck tested in the West Virginia study (2, Figure 8, which was reproduced as Figure 5.1 in the HCM). The equivalency factor values given by the manual were calculated by using the relative number of passings that would occur per kilometer (mile) of highway if each vehicle continued at its normal speed for the conditions under study. This definition is rather vague, giving no details of actual quantitative meaning, although a study by Werner (14) belatedly gives some details of the computational methodology employed. Since the adverse effects of trucks are quantified in terms of E_T, this section investigates E_T for several parameter combinations.

Twenty-four simulation runs were made for all combinations at all levels of the following four parameters:

1. Grades of 2, 4, and 6 percent;
2. Grade lengths of 1.9 and 5.6 km (1 and 3 miles);
3. Expected proportion of trucks of 10 and 20 percent; and
4. Expected mixed vehicle flow of 250 and 1000 vehicles/h.

An expected sample size of 200 primary direction vehicles was used in each run, as dictated by budget limitations. This in turn precluded analyzing situations with less than 10 percent trucks; the expected sample size of 20 trucks for this case was considered an absolute minimum for accuracy. The directional distribution of flows was equal. As previously mentioned, the 85th percentile overall travel speed obtained from the simulation was used in place of operating speed. The HCM is not exact in its definition of the length of road to which the adverse characteristics of truck-grade conditions are assumed to apply. For example, do the analysis procedures allow for the length of road required by trucks for acceleration following the top of the grade? In the present study, the travel speeds were calculated between the bottom of the grade and three points on the crest vertical curve (assuming a level road downstream of the grade):

1. The vertical point of curvature (VPC),
2. The point on the highway directly below the vertical point of intersection (VPI), and
3. The vertical point of tangency (VPT).

The points that gave the lowest 85th percentile travel speed were used in each case analyzed. In all runs a level warm-up subsection appropriate for the elimination of transient effects was used. After the warm-up subsection came the 1.6-km (1-mile) or 4.8-km (3-mile) grade as appropriate, followed by a 1.6-km (1-mile) level subsection to permit trucks to accelerate. The vertical geometrics of the simulated highways were selected according to American Association of State Highway Officials (AASHO) practices (15); this policy dictated the length of the no-passing zone at the top of each grade

Figure 7. Relation between capacity manual predicted v/c ratio and operating speed under truck-grade conditions.

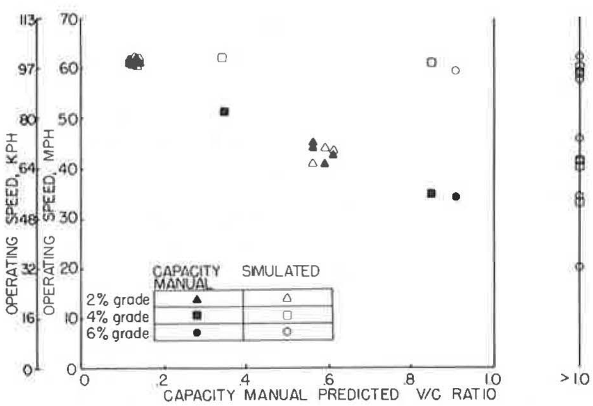
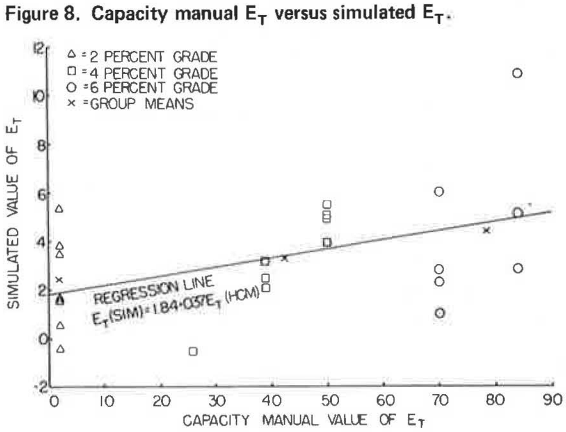


Figure 8. Capacity manual E_T versus simulated E_T.



and the overall sight distance. The horizontal geometry was assumed to be linear. The number of trucks used for these runs used an average distribution of types obtained from Figure II-10 of the AASHO policy (15).

These simulation results were compared with those shown in the HCM (Figure 10.2a), previously found to agree with the simulation under ideal conditions. Under truck-grade conditions, the HCM methodology inflates the actual flow of mixed vehicles to an equivalent flow, consisting of cars only, giving the same operating speed as the mixed flow. The v/c ratio corresponding to this inflated flow is found by solving equation 3, obtaining

$$v/c = SV/2000\ T_L \tag{4}$$

Thus, the relation between the operating speed and the v/c ratio still falls on the upper limb curves in Figure 10.2a of the manual. The 24 points on these curves corresponding to the various parameter combinations were calculated by using the FREHIS program (16), a strict computerization of chapters 9 and 10 of the HCM. The operating speed predicted by the manual was then simply read from Figure 10.2a by using the curve for the appropriate sight distance as restricted by the visibility over the crest. These points (for the cases in which the capacity was predicted not to be exceeded) are plotted in Figure 7. In addition, for each simulation run corresponding to a particular set of truck-grade parameters, one can use the manual's predicted v/c ratio (for the specified set of parameters) and the simulated 85th percentile travel speed as operating speed to locate a second point on the v/c-operating speed plane. If the simulation and the manual are in agreement, then this second set of points should also fall approximately on the curves of Figure 10.2a. This procedure has been followed in plotting the simulated points shown in Figure 7. Thus 24 comparisons are shown in Figure 7. For those cases in which the manual predicts congestion by yielding calculated v/c ratios in excess of 1.0, the operating speed, other than when the prediction is to be less than 56 km/h (30 mph), cannot be determined from Figure 10.2a. Therefore, only the simulated speeds are indicated for such cases, on a vertical line to the right of the graph. As is evident in Figure 7, except for the eight cases on the 2 percent grade where the agreement is close, the simulated results do not agree with the manual predictions. In fact, although the manual predicts congestion in 13 of the 16 cases for the 4 and 6 percent grades, the simulation predicts congestion for only one case.

In explaining the reasons for the discrepancy between the manual and simulation predictions, the truck equivalency factors E_T predicted by the model must first be determined and compared with the equivalency factors used in the manual. Therefore equation 3 for T_L is solved, yielding

$$T_L = SV/[2000\ (v/c)] \tag{5}$$

A new v/c ratio can be determined by entering Figure 10.2a with the operating (i.e., 85th percentile) speed observed in the simulation, and then reading v/c from the appropriate curve. SV is the expected flow of 250 or 1000 vehicles/h. Finally, the equivalency factor E_T can be found from T_L by using the relation given in the footnote of Table 10.12 of the manual. This procedure has been followed to obtain values for E_T for 23 of the 24 runs for which the simulated operating (85th percentile) speed was in excess of 56 km/h (30 mph). This method does not yield a solution for the one point for which the simulated operating speed is less than 56 km/h (30 mph). Thus, the 23 resulting simulated values for E_T are shown in Figure 8, plotted against the corresponding values from the manual. Although there is an ample amount of scatter in the simulated values of E_T, the group means and the regression line through the 23 points give an approximate idea of the comparison between the simulated and manual values for E_T. As can be seen, the simulated truck equivalency factors for the 2 percent guide are in general agreement with those in the manual. However, the simulated equivalency factors for the 4 and 6 percent grades are a whole order of magnitude less than those in the manual.

This discrepancy in the quantification of the effects of trucks can be ascribed to several causes. First, the truck decelerations on grades from the West Virginia study (2) used in Figure 5.1a of the HCM do not agree with those predicted by the Firey and Peterson model used in SIMTOL for the same 197.6-kg/W (325-lb/hp) mass-power ratio. For example, the Firey and Peterson model predicts a crawl speed of 70 km/h (38 mph) on a 2 percent grade, while Figure 5.1a gives a crawl speed of 41 km/h (22 mph). Similar results apply throughout the entire range of steeper grades; for a 6 percent grade, the values are respectively 13 and 26 km/h (8 and 16 mph). While the curves of Figure 5.1a have been obtained from the actual measurement of a test truck, the Firey and Peterson model has also been satisfactorily validated, first by the original authors (10 and 17) and later as a submodel in SIMTOL (3), as shown in Figure 3 in this paper. Because the Firey and Peterson model in SIMTOL is confirmed by more extensive field experimentation and validation, SIMTOL is being relied on; nonetheless, further investigation is required.

Second, the decision taken in the West Virginia study to consider a mass-power ratio of 197.6 kg/W (325 lb/hp) is questionable in light of present data. This value exceeds the median mass-power ratio for every truck class given in Figure 3 of Wright and Tignor (9) (Figure 5.3 of the HCM). Indeed, this value exceeds the maximum mass-power ratio for four of the six truck classes of that figure. While the selection of 197.6 kg/W (325 lb/hp) may be defended on the basis of conservatism, in the light of Wright and Tignor's work 197.6 kg/W (325 lb/hp) may be overly conservative for today's trucks. Work needs to be undertaken to obtain the current distribution of the mass-power ratio in the truck population.

There are two lesser reasons for the discrepancy. One is the vagueness of some portions of the HCM, particularly in the calculation of the truck equivalency factors; the computational methodology employed in this paper largely overcomes this difficulty. Vagueness also exists in the manual's definitions of operating speed and length of grade; the assumptions employed for this project have already been discussed and are not felt to play a significant role in the discrepancy of results. Finally, the noise in the simulated output, while certainly causing some random error, is not sufficiently great to account for the discrepancy. Not even the greatest extremities in Figure 8 approach the values used in the manual for the 4 and 6 percent grades; therefore the true expectation for these simulated points is far from the corresponding values given by the manual. More extensive simulation runs should be performed for the above as well as additional parameter combinations to more thoroughly quantify the adverse effects of trucks on grades.

The present method used by the HCM to quantify the adverse effects of trucks on two-lane, two-way highway grades overestimates the detriment of trucks to the traffic flow. Because this error is not great the conclusions should not result in any underdesigns. However, now when highway funding is dwindling, overdesign of highways must also be avoided. Therefore, the manual must be capable of reasonably predicting the actual traffic operations expected on a road. This study attempts not to offer revised truck equivalency factors for the manual but to point out some possible problems in the present factors so that these might be identified as a future research need.

CONCLUSIONS

This study should be of interest for two reasons. It introduces a useful new modeling tool, SIMTOL, for the evaluation of two-lane, two-way highways, and it brings attention to an area of the two-lane analysis procedure of the HCM that needs to be updated.

The model satisfactorily replicated actual human behavior in a series of validation runs. The model was applied to capacity studies of two-lane, two-way roads. Close agreement was evidenced between the model and the manual under ideal conditions, but the model predicts better traffic operations under truck-grade conditions than does the manual. This discrepancy is ascribed to several possible causes and emphasizes a need for revision of the effects of trucks as quantified in the manual.

Future work should investigate this weak area of the HCM addressed in the preliminary research presented here. Both modeling and empirical work need to be conducted. A particular need exists to expand the experimental design used here in the truck equivalency factor runs and to use larger simulation sample sizes and multiple realizations for each run.

ACKNOWLEDGMENT

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Comparison of Methods to Determine Intersection Service Level

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The most popular method of determining roadway capacity, the intersection analysis method, is described in the 1965 Highway Capacity Manual (HCM) (1). Although this method is simple and sound, it is not faultless. Problems in using the method have led to the development of other methods of capacity analysis. One such method is the critical movement summation (CMS) method presented by McInerney and Petersen in 1971 (2).

The following research compared the results of the two methods of service level calculation to determine if there is a significant difference. This study did not validate either one of the results, but did evaluate their correlation. Thus, the primary objective was to make paired observations of service level calculations by the CMS and HCM methods at selected at-grade intersections to determine the similarity of the service levels indicated. A second objective was to determine what sampling techniques, if any, were used to determine various factors required in the HCM calculation. Learning how widespread the knowledge of the CMS method is and what, if any, comparative opinions prevail was also desirable. The third objective was to determine the extent of use and acceptance of the CMS method.

METHOD OF TESTING

The procedure established to compare the results of the HCM method and the CMS method included sample selection, calculation procedures, index determination, and statistical procedures (3). The area selected for conducting the research was Montgomery County, Maryland. This highly urbanized county northwest of Washington, D.C., constitutes a significant portion of the Washington standard metropolitan statistical area.

The three basic types of intersections chosen were

1. Nonsignalized, type 1;
2. Signalized with simple two-phase operation, type 2; and
3. Signalized with a two-phase operation that allows at least one unopposed left-turn movement to take place, type 3.

Samples were taken from three ranges of service level: A to B, C to D, and E to F; the minimum sample size was 10 for each combination of intersection type and service level range.

The basic assumption of the CMS method is the thesis that there is a maximum range of traffic volume that can move through the critical lanes during each signal cycle. This research used a modification of the CMS method that was developed by the Montgomery County Department of Transportation and adopted by the Maryland-National Capital Park and Planning Commission (4). The following critical lane volumes were selected from the Montgomery County modification as being representative of the various levels of service.

Level	Vehicles per Hour	Level	Vehicles per Hour
A	<1000	D	1301 to 1450
B	1001 to 1150	E	1451 to 1600
C	1151 to 1300	F	>1600

These volumes are based on the following statement in the HCM (1, p. 126): "If all approaching vehicles in the traffic flow are stopped on the approach before entering, then rarely can traffic move away at a rate greater than 1500 vehicles per hour of green per lane."

Assuming an average of 10 percent cycle time used for amber signals (normal for most simple intersection phasing), a volume of 1500 vehicles/h was reduced to 1350 vehicles/h. This volume indicated service level D, somewhat below capacity. The following procedure was then used to calculate critical lane volume:

1. The existing traffic volume was determined for each movement permitted at an intersection during the hour to be evaluated;
2. The volume of total traffic by direction moving during each phase of the signal operation was tabulated (two-phase was assumed for nonsignalized intersections);

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ıty of Service.

3. Each approach was adjusted by appropriate lane use factors;

4. If an unprotected (not separately phased) left-turn movement had performed in conflict with through traffic, this conflict was added to the adjusted through volume in the critical opposing lane because this traffic also used the same phase;

5. Careful attention was given to the particular phasing to ensure that the traffic volume operating during more than one phase was properly distributed; and

6. The maximum volume flowing during each phase was computed, the highest volume was selected as the critical lane volume, and the appropriate level of service was selected.

The HCM method for determining an at-grade intersection service level is more complex. Service volume at capacity (i.e., load factor equals 1.0) is calculated by using tables and figures from chapter 6 of the manual and is compared with the real volume for each approach to the intersection. This volume/capacity (v/c) ratio can be compared to the range of ratios indicated in Figure 10.3 of the manual and an appropriate level of service can be selected.

Since the sample size was small and the distribution was abnormal, a nonparametric test was necessary and, therefore, the Wilcoxon Test for Paired Observations (5) was used.

DATA COLLECTION AND COMPUTATION

Typical data requirements for each hour of traffic conditions include physical conditions (i.e., dimensions), traffic volume characteristics, and signal phasing and timing. The primary source of data was the Division of Traffic Engineering, Montgomery County Department of Transportation. Some data were also received from the Maryland Department of Transportation.

Sample selection began with critical movement summation analysis of type 1 intersections; physical condition and available turning movement counts were used. An attempt was made to find intersections that had CMS service levels in all three ranges. Samples in the E to F range, however, could not be located, and only two samples in the C to D range were found. Ten samples were found for the A to B range. Only five samples of type 2 and nine samples of type 3 intersections were found in the E to F range (3). For type 2 and type 3 intersections, ten samples each were found for the C to D and E to F ranges.

QUESTIONNAIRE USED TO ESTABLISH GUIDELINE FOR HCM CALCULATIONS

Among the objectives was the development of sampling techniques to determine factors needed to apply the HCM method. Calculation of service level by the HCM was desired to be similar to calculation used in real practice. Another objective was learning the extent of exposure that the CMS has and its ease in application and communication. To accomplish these objectives a questionnaire was prepared and distributed.

Four questions in the questionnaire dealt with the methods used for capacity analysis or service level determination. The fifth question was used to establish a guideline for HCM calculations that precede the basic objective of this study. The following is a summary of the findings of question 5 (3).

1. Bus factor is based on judgment.
2. Truck percentage is based on a single sample and judgment.
3. Turning percentage is based on a single sample.
4. Load factor is based on a single sample, judgment, and area mean.
5. Peak-hour factor is based on a single sample, judgment, and area mean.

STATISTICAL RESULTS

The desired sample size for type 1 intersections was available only in the A to B service range. The HCM method indicated 9 to 10 samples had inferior levels of service.

The desired sample sizes for type 2 intersections were obtained for A to B and C to D service level ranges, and a limited sample was obtained for the E to F range. In both A to B and C to D ranges, the levels of service produced were lower by the HCM method than by the CMS method. Results of both methods compared favorably in the E to F range; however, the limited sample size decreases the credibility of these data.

The desirable sample size was obtained for all three service levels at type 3 intersections. Again the HCM method generally indicated levels of service lower than those indicated by the CMS method. However, the correlation of results improved in the E to F range.

FINDINGS

This study (3) netted the following conclusions.

1. The HCM method and the CMS method produced different results for intersection analyses as indicated by the Wilcoxon statistical test.
2. The service levels calculated by the HCM method for intersections are usually inferior to those calculated by the CMS method.
3. The service levels calculated by the HCM and CMS methods appear to be closer when the intersection is signalized and flowing at level E or F.
4. Use of the questionnaire revealed that (a) the majority of the respondents used the HCM method or related charts for at-grade service level determination; (b) the sampling techniques used by most respondents for determining various HCM factors were single sample and judgment; (c) exposure to the CMS method is very limited and exists primarily in the eastern part of the United States; and (d) those who use the CMS method judge it easy to use and to communicate.

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Measuring Delay by Sampling Queue Backup

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The relation of sampling of queue backup and delay at signalized intersections was studied and evaluated for use as a level-of-service indicator for intersection performance. Time-lapse photography was used at four urban intersections controlled by pretimed signals to determine time-in-queue delay. At the same time, each field observer sampled the position of the rear of the queue in one lane at 10-s intervals. Other field methods of measuring delay were tested also. Regression analyses of resulting delay values by cycle yielded high correlations between queue backup delay from field sampling and time-in-queue delay from film analysis. Field sampling of queue backup was found to be much simpler to use in the field than field sampling of stopped time delay. Field sampling was confined primarily to three unsaturated approaches that had few left-turning vehicles. Further study is needed to validate and refine field procedures under a wider range of conditions.

This report summarizes results of studies of methods for field measurement of delay at signalized intersections (1). The purpose of the study was to identify and validate a method that traffic engineers can use in the field to measure intersection performance. The method should be simple enough for widespread use in obtaining delay; updated data are particularly needed to revise the chapter on signalized intersections of the Highway Capacity Manual (HCM) (2).

Figure 1 illustrates the three types of delay at approaches to signalized intersections that have been identified in previous research and are discussed below.

1. Travel-time delay (TTD) is the difference between the time a vehicle passes a point downstream of the intersection where it has regained normal speed and the time it would have passed that point had it been able to continue at its approach speed.
2. Stopped-time delay (STD) is the time a vehicle is substantially standing still while waiting in line in the approach to a signalized intersection (3).
3. Time-in-queue delay (TIQD) is the difference between the time a vehicle joins the rear of a queue and the time the vehicle clears the intersection (4). Other authors have used different terms, specifically, queue delay (5), aggregate delay (2), and system delay (6).

Methods for field measurement of delay that have been studied previously include

1. The Berry-Van Til method, in which stopped-time delay is periodically sampled (2, 3, 7, 11),
2. The Sagi-Campbell method for determining TIQD, in which queue lengths are observed at specified times in each cycle (8),
3. The delay meter method, in which vehicles are input as they stop and output as they enter the intersection and the meter accumulates the TIQD (2, 3, 10),
4. The volume density method for determining TTD, in which observers count the number of vehicles occupying the section of the approach under study at successive time intervals such as 15 s (9), and
5. The time-lapse photography method that is used primarily to validate other field methods that utilize observers (2, 3, 5, 11).

To investigate the current use of delay as an indicator of signalized intersection performance, a questionnaire was sent to 78 traffic engineers with experience in delay measurement. The questionnaire attempted to determine which types of delay are thought to be most useful as indicators of intersection performance, which types of delay are considered the easiest to measure in the field, and which types of field methods for measuring delay are being used most often. Questions regarding field techniques also were asked to facilitate selection of standards for data collection in the field.

Forty traffic engineers or 51 percent of the sample responded. Tabulated replies to questions relating to delays considered "most useful" and "easiest to measure" are shown in Table 1. Each respondent indicated first preference as Rank 1, second preference as Rank 2, and third preference as Rank 3.

Weighted totals of the most useful indicators show that TTD is ranked first and STD second. Weighted totals of the easiest-to-measure indicator reveal that average length of queue ranks first, STD ranks second,

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and load factor ranks third.

The field method reported as most frequently used was measurement of STD. Method TTD was reported the second most frequently used. Most (26 of the 39 who responded to the question) measured intersection delay over the entire approach width rather than by separate lanes. In reporting on criteria for when a vehicle was considered delayed, 11 of 28 used a locked-wheel criterion, but 17 others reported that vehicles are considered delayed when speeds are below 3.2, 4.8, or 8.0 km/h (2, 3, or 5 mph).

RELATIONSHIPS OF INDIVIDUAL VEHICLE DELAY TYPES

Three types of vehicle delay—TTD, STD, and TIQD—are obviously interrelated, as shown in Figure 1. Equations were developed for computing the differences in delay, assuming that each delayed vehicle decelerates to a stop at a uniform rate and then accelerates to its departure speed at a uniform rate of acceleration (1).

The equations reveal that TTD always exceeds STD by the amount of time spent accelerating and decelerating and that the difference is greater for higher initial speeds. In contrast, TTD is greater than TIQD only when the vehicle stops close to the stop line, but the reverse is true when the vehicle stops farther from the stop line. The results suggest that average TIQD can be expected to be closer to average TTD than average STD is.

METHODS USED

A base method for obtaining delay in each lane was applied by using time-lapse photography. During filming observers used at least two different manual methods for field measurement of delay. Results of manual methods were then compared with results from the base method by use of regression analysis on delays as calculated per cycle.

Base Method

TIQD was selected as the base method for the following reasons.

1. TIQD for each vehicle can be measured quickly and accurately by using time-lapse photography.
2. TIQD approximates TTD, which is more difficult to measure accurately even with time-lapse photography because of different approach speeds and varying speed change rates of different vehicles.
3. TIQD appears to approximate the individual driver's concept of delay, since TIQD covers the time utilized from the stop until the driver is sure of clearing the intersection.

As many as three cameras were used simultaneously from different camera positions to identify queue positions accurately. Film speed was 1 frame/s except for one series where the speed was 5 frames/s. Time was estimated to the nearest 0.5 s for a film speed of 1 frame/s. These film speeds were considered to be adequate, since a vehicle at 4.8 km/h (3 mph) travels only 1.3 m/s (4.4 ft/s).

Manual Methods

Observers used two field methods for measuring delay: average length of queue and TIQD sampling.

Average Length of Queue

In the average queue length method, or the queue backup sampling method, the position of the rear of the queue is sampled at predetermined intervals, such as every 10 s. The time interval is referred to as the sampling interval. The position of the rear of the queue is recorded either in number of vehicles or in distance.

The queue position in number of vehicles includes all vehicles that previously stopped during the formation of the queue. Thus, from Figure 2 the queue position at sampling time 40 is recorded as 9 even though seven of the vehicles have already begun moving.

At each sampling time, adjustments are made for lane changing. Field observers note when a vehicle enters or leaves a lane via another lane or driveway and records the appropriate interval. Field notes for different lanes are later cross-checked and compared for the balance in lane changing. In treatment of loaded cycles, care is taken to double-check the number of vehicles accumulated in the queue at the beginning of the red to avoid cumulative errors.

When queue position is recorded in distance, the distance from the rear of the last queued vehicle to the stop line (entry to the intersection) is recorded in meters (feet). Lane changing is not recorded.

The position of the last queued vehicle at each sampling time is converted into TIQD by multiplying the total number of queued vehicles by the interval size. For some observations, portable tape recorders provide signals to tell observers when to sample queue lengths. In other field studies, special buzzer and bell signaling devices provide the needed signals.

In addition to sampling queue backup at 10-s intervals, recording the approximate time the last vehicle in the queue started for each cycle is possible. This queue-max method of sampling queue backup was tested by using film data and is explained later.

TIQD Sampling

With the TIQD sampling method, different observers simultaneously count during predetermined time intervals the number of vehicles joining the queue and the number of vehicles entering the intersection. The number of vehicles each observer counts during the interval is recorded at the end of the interval. Vehicles that enter or exit the queue via off-street side areas or other lanes must be counted either as in or out during the correct interval. The TIQD sampling method is similar in concept to the delay meter method (10).

Because of limited personnel available for field studies, only limited testing of this field method was possible; a 10-s sampling interval was used for 56 cycles of 110 s each.

Preliminary Field Studies

Preliminary field studies were undertaken to check criteria for determining when vehicles joined the queue; time-lapse photography and four field observers were used. For these studies a speed of approximately 4.8 km/h (3 mph), considered normal walking speed, was selected as a standard because observers in the field could most easily judge when a vehicle had slowed to that particular speed. The 4.8-km/h (3-mph) value was also consistent with the 3.2 to 8.0-km/h (2 to 5-mph) range indicated by the questionnaire as being most commonly used.

In these field studies, the observers, who were physically separated from one another, were told to raise a white placard when they filt that the next car joining their queues had reached the 4.8-km/h (3-mph) speed. A time-lapse film taken of the observers permitted calculation of the time that each observer believed that the TIQD should begin for each vehicle. The film also was used to get the time each car crossed the stop line. The TIQD was then calculated for pairs of observers for each lane. Statistical tests were run on the mean delays of the lane-observer pairs. Results indicated that the average delays were not found to be significantly different at the 5 percent significance level (1). Accordingly, the 4.8-km/h (3-mph) standard was used in the rest of the study.

In the preliminary studies, observers also checked different systems for reminding observers when to sample queue lengths. Stopwatches, bell and buzzer systems, and portable tape recorders were tested. Portable tape recorders that signaled the sampling time interval periodically were found to be satisfactory, provided adjustments were made occasionally to keep the signal in synchronization with the cycle.

A 10-s sampling interval was used by observers in the field studies, since all intersections studied had pretimed signals with cycle lengths in multiples of 10 s. Thus, queue lengths were sampled at the same relative sampling points in each cycle. This type of sampling may introduce some systematic error as discussed later, but facilitates the computation of delay by cycle, which may be needed when the HCM is used for level-of-service determinations.

Another preliminary study investigated use of rubber cones and other methods for sampling the position of the rear of the queue by distance rather than by the number of cars in the queue. Such methods would be especially useful when the queues are very long. Results of these studies for a limited number of observations indicated that, for queues of passenger cars, average distance headway was 8.23 m (25.1 ft), average spacing between cars was 2.95 m (9.0 ft), and the standard deviation was 1.03 m (3.14 ft).

Figure 1. General relationships between three types of individual vehicle delay.

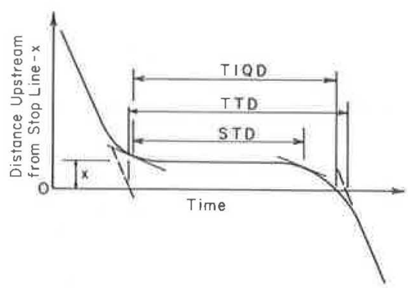
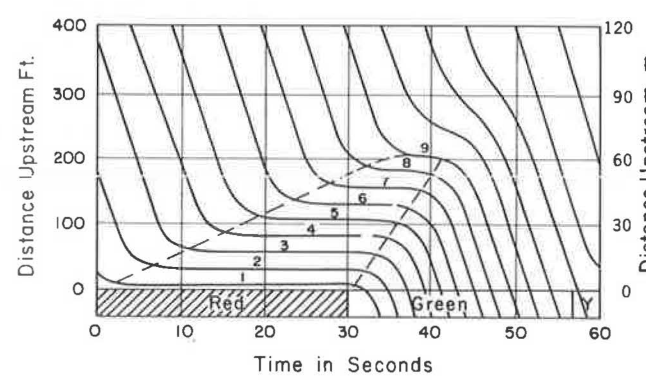


Figure 2. Time-space diagram based on 75 percent of vehicles stopping for signal.



COMPARISON FOR ONE CYCLE

Figure 2 shows a time-space diagram that is helpful in identifying how measurements are made for various methods of determining delay. TIQD, TTD, and STD have been scaled from this diagram for each of the delayed vehicles in this cycle as shown in Table 2.

Similarly, queue backup and STD have been sampled for each of the 10-s sampling points in this cycle. For computational purposes the sampled queues are considered to extend one-half the interval size either side of the sampling point. For computation of delay per cycle, queues recorded at the cycle changes are multiplied by one-half the interval size. Shown in Table 3 are sampling results, including use of a modified queue backup sampling, in which the maximum queue length and the time it occurred are also recorded and used in computations of queue backup delay.

Time-in-queue sampling by counting input to the queue and output from the intersection at 10-s intervals is illustrated in Table 4; data from the time-space diagram are used.

These examples do not illustrate the handling of vehicles changing lane, vehicles entering the queue from driveways, or loaded cycles. Procedures for adjusting the counts for these conditions were mentioned earlier.

DATA COLLECTION

Field data were collected both by time-lapse photography and by field observers for three urban intersection approaches, controlled by pre-timed signals, in Chicago and in Evanston. In addition, data taken by time-lapse photography for a fourth intersection were analyzed to provide some validation of sampling methods for loaded-cycle conditions. Characteristics of these four intersections are given in Table 5.

All data were taken for a peak direction of flow between 4:00 and 6:00 p.m. Three intersection approaches had no loaded cycles; however, the fourth had 57 percent

Table 1. Number of respondents by ranking of indicators of signalized intersection performance.

Type of Delay or Other Indicator	Most Useful				Easiest to Measure			
	Rank 1	Rank 2	Rank 3	Weighted Total	Rank 1	Rank 2	Rank 3	Weighted Total
TTD	16	3	4	58	5	4	2	25
TIQD (via meter)	6	12	5	47	0	8	6	22
STD	7	11	7	50	8	4	5	37
Average length of queue	2	6	7	25	8	8	4	44
Maximum queue per cycle	2	2	1	11	5	5	5	30
Vehicles stopped, %	0	2	4	8	2	7	3	23
Load factor	2	2	1	11	10	1	3	35

Table 2. Delays scaled from time-space diagram.

Vehicle Number	Delay (s) Base TIQD	TTD	STD	Vehicle Number	Delay (s) Base TIQD	TTD	STD
1	30	34	29	8	15	12	8
2	28	30	26	9	13	9	5
3	26	27	23	10	–	6	–
4	24	24	20	11	–	4	–
5	22	22	17	12	–	3	–
6	19	18	14	Total	194	204	153
7	17	15	11				

Table 3. Queues sampled from time-space diagram.

Type	Sampling Time 0 s	10 s	20 s	30 s	40 s	50 s	60 s	Max Queue Vehicles	Time (s)	Total Delay (vehicle-s)
Queue backup delay	0	2	5	7	9	0	0	–	–	230
Queue backup with max queue[a]	0	2	5	7	9	0	0	9	42	203
Stopped time delay	0	2	5	7	2	0	0	–	–	160

[a](10/2) (0) + 10 (2 + 5 + 7) + (42 - 35) 9 + 10 (0) + (10/2) (0) = 203 vehicle·s.

Table 4. Time-in-queue sampling from time-space diagram.

Item	Sampling Interval (s) 0 to 10	10 to 20	20 to 30	30 to 40	40 to 50	50 to 60	Total
Vehicle input	2	3	2	2	0	0	9
Vehicle output	0	0	0	4	5	3	12
Accumulation of vehicles	2	5	7	5	0	0	19

Note: Total cycle delay = 19(10) = 190 vehicle · s.

loaded cycles. Delay data for each lane were taken from the films. Included for each vehicle were the frame numbers (a) when the vehicle slowed to approximately 4.8 km/h (3 mph) at the rear of the queue, (b) when its rear wheels crossed the stop line at the intersection, (c) when it, as the last vehicle in the queue, began moving, and (d) when any lane changing occurred. These data, as well as the delays computed by the field methods, were punched on cards for computer analysis. All data were tabulated and analyzed by lane and by cycle.

When data for loaded cycles were taken from films, some individual vehicles were stopped in more than one cycle. The portion of the delay occurring in each cycle was assigned to that cycle.

REGRESSION ANALYSIS

A linear regression model was used to test the correlation between the base TIQD and the delay obtained by field methods for sites A, B, and C. Table 6 lists regressions plus the number of cycles (data points) included in each regression.

Equations resulting from the regressions are given below, as are values of correlation coefficients and the number of data points in each regression. These regression equations should not be interpreted to mean that accuracy is greater than would be expected from sampling data to the nearest 0.5 s.

Base TIQD = 4.906 + 0.937 (queue backup sampling) (1)

Base TIQD = -18.704 + 1.079 (queue backup sampling) (2)

Base TIQD = 18.230 + 1.030 (TIQD sampling) (3)

Base TIQD = 0.647 + 0.978 (queue backup with maximum queue) (4)

Regression equation 1 yielded a correlation coefficient of 0.971, indicating a high degree of correlation. The slope of 0.937 was found to be significantly different from 1.0 at the 5 percent significance level, but not at the 1 percent level.

Regression equation 4 yielded a slope of 0.978, an intercept of 0.647, and a correlation coefficient of 0.982. The slope was not found to be significantly different from 1.0 at the 5 percent level.

Regression equation 3 also yielded a high correlation coefficient of 0.966. The slope of 1.030 was not found to be significantly different from 1.0 at the 5 percent significance level.

A regression also was prepared to correlate base TIQD with queue backup delay based on queue length rather than number of vehicles in the queue.

Base TIQD = 65.098 + 0.799 (queue backup: distance) (5)

This sample for 20 cycles at site B yielded a correlation coefficient of 0.906, somewhat poorer than that yielded when number of vehicles is used for queue backup sampling. However, further tests are needed of use of distance as a measure of queue backup, since use of distance sampling is about the only practical method when queues are very long.

Regression equation 2 for only 20 cycles at a through plus left turn lane at site A yielded preliminary information that queue backup sampling is useful in measuring delay for such lanes (correlation coefficient of 0.947); more study is needed for locations with heavier turning movements and more opposing flow.

These regression results indicate that sampling queue backup is very promising for use when data are collected for revising the signalized intersection chapter of the HCM. The simplified queue backup method correlates reasonably well with base TIQD. Use of the modified queue backup sampling method, plus additional data on when the maximum queue length occurs, should improve the correlation with base method TIQD.

The time-space diagrams of 15 cycles for each of the two lanes of the south approach of Green Bay Road at Central Street, Evanston (site D), had been plotted by Centeno (7), based on time-lapse photography taken at

Table 5. Characteristics of four study sites.

Site	Intersection	Cycle Length (s)	Red Phase (s)	Northbound Lanes	Lanes Studied	Turns Right	Turns Left
A	Sheridan at Noyes, Evanston	90	30	2	2	None	Yes
B	Sheridan at Foster, Evanston	90	32	2	1	None	None
C	Sheridan at Glenlake, Chicago	110	22	3	Middle and center	None	None
D	Green Bay at Central, Evanston	90	60	2	2	Yes	Yes[a]

Note: No parking was permitted at study sites.
[a]No opposing flow during northbound phase of three-phase signal.

Table 6. Regressions and number of cycles for three sites.

Regression Number	Type of Lane	Number of Cycles Used: Site A	Site B	Site C (per lane)	Total
1 and 4	Through	20	20	28	96
2	Left	20	—	—	20
3	Through	—	—	28	56

5 frames/s. Base TIQD values for the 30 cycles varied from 87 to 1071 vehicle-s/lane/cycle. These time-space diagrams provided an opportunity to apply the TIQD sampling and queue backup methods to loaded cycles. A sampling interval of 10 s was used for these data also. Each of the two lanes of this approach was treated separately. For curb-lane traffic, 6 of 15 cycles were loaded; for center-lane traffic, 11 cycles were loaded. Left turns made from this center lane were unopposed since this intersection is controlled by a three-phase signal.

The regression of the base TIQD on the TIQD sampling method for the curb lane produced the equation

$$\text{Base TIQD} = 3.668 + 0.957\ (\text{TIQD sampling}) \tag{6}$$

The regression for the center lane yielded

$$\text{Base TIQD} = 32.945 + 0.933\ (\text{TIQD sampling}) \tag{7}$$

The correlation coefficients obtained for each of the lanes were almost identical: 0.968 for the curb lane and 0.967 for the center lane. Significance tests run on the slopes and intercepts revealed that the intercepts were not found to be significantly different from zero and that the slopes were not found to be significantly different from one at the 5 percent significant level.

Similar regressions were run for the queue backup method, yielding the following:

$$\text{Base TIQD} = 67.512 + 0.860\ (\text{queue backup}) \tag{8}$$

for the curb lane and

$$\text{Base TIQD} = 7.652 + 1.007\ (\text{queue backup}) \tag{9}$$

for the center lane. A regression run for 30 data points for the two lanes combined yielded

$$\text{Base TIQD} = 53.135 + 0.883\ (\text{queue backup}) \tag{10}$$

The correlation coefficients were 0.967, 0.958, and 0.970 respectively for the three equations. These results indicate that these methods could work well for loaded conditions. Supposition is indicated since the data were not collected by field observers but were taken from time-space diagrams. Caution should be exercised, therefore, in viewing the results reported as indicative of the method's accuracy when field observations are taken under saturated traffic flow conditions in which lane changing occurs.

Centeno (7) attempted to make stopped-time delay observations in the field to correlate with film data for peak-hour conditions at Green Bay Road and Central Street. However, he found that he could not sample stopped-time delay for the long queues with some exceeding 194 m (636.5 ft). Samples taken after the beginning of the green require simultaneously observing the back of the queue and identifying how many vehicles have started near the front of the queue. Accordingly, he limited himself to analyzing data taken from films.

LENGTH OF SAMPLING INTERVAL

To determine whether varying the size of the interval used would drastically affect the results obtained with queue backup sampling, a computer program was written in which the individual vehicle data taken from the films were used to compute the total delay per cycle. The computer program used only intervals that were evenly divisible into the cycle length. Accordingly, the analysis was performed separately on the data from the 90-s and the 110-s cycles. For the 90-s cycle, intervals of 5, 6, 9, 10, 15, 18, and 30 s were used; for the 110-s cycle, intervals of 5, 10, 11, and 22 s were used.

A regression equation was developed between the delay for the queue backup method, calculated from field data, and the delay for 10-s sampling intervals, calculated by computer program. Based on the data from the through lanes of sites A, B, and C, the relationship developed was

$$\text{Film queue backup} = 4.437 + 0.999\ (\text{field queue backup}) \tag{11}$$

The number of cycles was 95, and the correlation coefficient was 0.961. The intercept and the slope were not found to be significantly different from zero and one respectively at the 5 percent significance level. The indication then is that the standard for when a vehicle was considered stopped was uniformly applied in the field and when data were taken from the films.

Regression equations then were developed for each sampling interval to compare the base TIQD with the queue backup sampling from film data. The data from the first two sites (90-s cycles) were combined, giving a total of 60 cycles. The two lanes of the third site (110-s cycles) were also combined, resulting in a total of 56 cycles.

For the data from the 90-s cycle, the coefficient of correlation values were above 0.96. However, the intercepts and slopes of the equations were not found to be statistically different from zero and one respectively at either the 5 percent or the 1 percent significance levels for all intervals of 15 s or lower (5, 6, 9, 10, and 15 s). For the data from the 110-s cycle, the coefficients of correlation were above 0.96 only for intervals below 12 s. The 22-s interval equation gave a correlation coefficient of 0.86. The intercepts of the 5, 10, and 11-s interval equations were not found to be statistically different from zero at the 5 percent significance level. The slopes were found to be significantly different from one for all equations at the 5 percent significance level. Thus, the queue backup method does not give good results at large sampling intervals.

The results seem to indicate that the queue backup method should be used with small sampling intervals. Errors involving the maximum queued vehicle may be significant when the queue is long. Overcoming this effect may be possible by recording, in the field, the time the maximum queued vehicle starts forward. Getting this time exactly may be difficult; however, if a readily visible timing device were used, the observers may be able to estimate the time the queue ends to some fraction of the interval. Such a timing device for obtaining this information might be a 10-s stopwatch that emits an audible signal every 10 s and could be attached to the observer's clipboard.

An attempt was also made to determine whether the use of a nonintegral 11-s sampling interval significantly affects the results obtained with the queue backup method. The data selected to perform this analysis were taken from the time-lapse films of the traffic at site B, Sheridan Road and Foster Street. Delay was computed by cycle for each of the 40 cycles. Regressions of the base TIQD on the queue backup delay were performed for results obtained by using both the 10-s integral and the 11-s nonintegral sampling intervals. The regression equations are shown below.

$$\text{Base TIQD} = 23.089 + 0.923\ (\text{queue backup}) \tag{12}$$

for the 10-s interval and

$$\text{Base TIQD} = 16.607 + 0.949\ (\text{queue backup}) \tag{13}$$

for the 11-s interval. The correlation coefficients obtained from each of the regressions were 0.965 and 0.946 respectively. Thus, the delay computed by using the queue backup method is highly correlated with the true values for both the 10-s and 11-s sampling intervals.

This limited analysis indicates that the choice of a sampling interval for use in the queue backup method can be either an integral or nonintegral subdivision of the cycle length for intersections similar to those studied here. Results obtained by using either of these interval types should not be significantly different from one another, provided that the sizes of the intervals are of the same order of magnitude.

PERSONNEL REQUIREMENTS

Queue backup sampling normally requires one person per lane for long queues. When queues are not long, or when the number of queued vehicles is approximately the same per lane, one observer is sufficient for the simplified method of queue backup sampling. When observations are also made of the time the maximum queued vehicle starts as part of queue backup sampling, the field work is more difficult. More study is needed of this maximum-queue option in queue backup methodology, especially for lanes where left turns are permitted and vehicles change lanes.

If the observer at the back of the queue observes queue position in meters (feet) rather than in number of vehicles in the queue, he or she usually can collect data on the queue positions for each of two lanes. Observers can devote all of their time to identifying the position of the rear of each queue.

Time-in-queue sampling requires more care in field application to avoid error from vehicles changing lanes and entering from driveways and other side entrances. If there are many lane changes, one observer may be needed to handle lane changes only. In studies of intersection performance for intersection capacity purposes, personnel counting entering volumes per cycle might be able to collect the output count data for the time-in-queue sampling; thus, only one additional person per lane would be needed at the rear of the queue to implement this method of measuring delay.

In general, the less complex queue backup method is simpler to use in the field than the sampling of stopped-time delay method because the observer need not count vehicles that are starting at the front of the queue.

CONCLUSIONS

This investigation of field methods for measuring TIQD used only a limited number of intersection approaches. The following conclusions, therefore, apply primarily to approach lanes with traffic and physical conditions similar to those studied.

1. Pending further validation under a wide variety of conditions, queue backup sampling appears to be very promising for field use in a nationwide effort to measure delay at signalized intersections because the number of personnel and amount of equipment required are minimal, data collecting is simple, and the method has reasonably close correlation to TIQD. The method gives evidence of being compatible with other data-gathering procedures for analyzing intersection capacity and performance by cycle.
2. Both queue backup sampling and TIQD sampling measure TIQD in the field reasonably well for the conditions encountered in this study. These conditions included few left-turning vehicles, little lane changing, and unsaturated traffic flow. Accuracy of queue backup sampling can perhaps be improved by collecting data on the starting times of the last queued vehicle in each cycle.
3. Simplified queue backup sampling requires fewer personnel and is easier to perform in the field than TIQD sampling under most conditions. However, if the additional data obtained by the TIQD sampling method (i.e., saturation flows or entering volumes) are needed for capacity studies, using this method warrants further consideration.
4. In queue backup sampling by distance, somewhat poorer correlations with base TIQD were yielded when the position of the rear of the queue was sampled in meters (feet) than when counts were made of the actual number of vehicles in the queue. However, the limited experience with this distance-sampling method indicates that the method is easy to use in the field because there is no need to count vehicles and to keep track of lane changing.
5. The sampling interval selected for use in the field in the queue backup method may either be an integral or nonintegral subdivision of the cycle length. Limited tests conducted with these two different sampling rates yield similar results. Also, the accuracy of the results using the queue backup method appears to be dependent on the selection of a relatively small sampling interval, such as 10 s.

RECOMMENDATIONS FOR FURTHER RESEARCH

The methods used in this study to measure TIQD were field tested under limited conditions. To adequately evaluate the potential of these methods for use in further studies to replace load factor as an indicator of signalized intersection performance, several conditions should be investigated.

1. Further field testing should be conducted of queue backup sampling methods under saturated flow conditions to better determine the method's abilities to predict TIQD under loaded conditions. Since these heavy-volume conditions are often of most interest, this recommendation should receive high priority in further research.
2. Further field testing is necessary for left-turn lanes, where the length of the queue continually varies because of the blocking effects of left-turning vehicles and vehicles changing lanes, to better determine the predictive ability of the method under such conditions.
3. The ability of field personnel to record lane changes accurately under various traffic conditions should be field tested to determine whether this possible source of error in sampling the number of vehicles in the queue can be minimized.
4. A further investigation of queue backup sampling should be conducted in which the position of the last vehicle in the queue is recorded as a distance upstream from the intersection. Different types of markers alongside the approach roadway should be used to facilitate estimation of distances. Refinements should be made in the field procedures for observing the actual position of the rear of the queue at each sampling time and in the application of the method to lanes with left-turning vehicles and with commercial vehicles. Queue lengths could be identified by distance markers; each marker might correspond to a number of equivalent passenger vehicles. Delay could then be computed in seconds per equivalent passenger vehicle. This method would avoid calibrating queue lengths due to variations in number of commercial vehicles.
5. The use of a maximum queue correction should be investigated in the field for the queue backup method based on the length of maximum queue (in numbers of vehicles or in distance) and the time the last queued vehicle starts. Such an investigation should determine (a) the ability of field personnel to collect these data and (b) whether these maximum queue data improve the ability of queue backup methods to predict TIQD sufficiently to warrant any extra personnel.

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Relationship of Signal Design to Discharge Headway, Approach Capacity, and Delay

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The rate at which a queue at a signalized approach discharges vehicles has a major effect on the capacity of the approach. To determine the relative effectiveness of various signal configurations and lens sizes in dissipating queues, queue discharge headway measurements were made at 38 sites in five states. Four major classes of signal configurations and two lens sizes were analyzed. The results show that, except for lens size, no class of configurations can be considered better than any other class for any queue position. Estimates are made of expected delay and approach capacity as functions of configuration class and lens size.

Traffic control signals represent an unavoidable impedance to traffic flow. No matter how well a signal is timed and no matter how well that timing is maintained and adapted to instantaneous conditions, some vehicles arrive during the red interval. These vehicles form a queue that must be dissipated during the ensuing green interval. The rate of queue dissipation has a major effect on the capacity of the approach; also, the rate of queue dissipation greatly determines smoothness of flow of the primary platoon.

As part of a major study of traffic signal design configuration (NCHRP Project 3-23, Guidelines for Uniformity in Traffic Signal Design Configurations) queue discharge behavior was investigated. A nationwide program of empirical testing was undertaken to determine the absolute values of the queue discharge headway distribution and the influence of various factors on this distribution. The main emphasis was to determine how number, size, and location of traffic signal heads could be changed to improve the queue discharge headway distribution.

DATA COLLECTION

Characteristics of Data Collection Sites

The sites at which queue discharge headway data were collected are listed in Table 1. The signal configuration number refers to a combination of number, location, and mounting height of traffic signal head. Many combinations are possible, and nearly 50 have been identified (1). The 18 signal configuration numbers used in Table 1 are sketched in Figure 1.

Data Collection and Reduction

Queue discharge headway data were recorded by manual input to a chart recorder. The observer pressed a button when the signal changed to green and when a vehicle passed the stop line (or a screen line established as the location of the front wheels of the first car in queue). Data were recorded for all passenger cars on each cycle that

1. Were stopped in queue at the beginning of the green interval;
2. Proceeded straight through the intersection; and
3. Were not impeded by pedestrians, cross traffic, or opposing left turns.

Data were collected for approximately 30 cycles at each location. Figure 2 shows a typical recorder chart.

This manual input method has an element of error because of the observer's reaction time. To compensate for this error the reaction time was assumed to be almost uniform for all the inputs. This assumption was validated by film. Queue discharge data were manually collected at one location by using a chart recorder while, at the same time, the queue discharge process was filmed at 5 frames/s. Both sets of data were reduced, and the queue discharge distribution was determined. No significant differences between the two distributions were detected.

The filmed data were reduced on a frame-by-frame basis. The queue discharge headway data recorded on the charts were reduced by measuring the times between the onset of green spike and the first vehicle spike. Then the time between each succeeding vehicle passage was measured. The sample size at each queue position decreased from approximately 30 at the first position to zero at more distant queue positions.

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Table 1. Data collection sites and characteristics.

Number	Location	Intersection	Approach[a]	Equivalent Lanes	Median[b]	Left Turn Lane[c]	Control Type[d]	Lens Size[e]	Signal Configuration Number	Posted Speed Limit (km)	85th Percentile Speed (km)
1	Somerville, Mass.	Highland and Lowell	NB	1	0	0	1	3	13	48	43
2	Brookline, Mass.	Beacon and Dean	EB	2	1	0	1	2	13	48	62
3	San Francisco	California and 25th	WB	1	0	0	1	1	13	40	50
4	San Francisco	California and 25th	EB	1	0	0	1	1	13	40	48
5	Lansing, Mich.	Michigan and Penn	WB	2	0	1	1	1	4	40	50
6	East Lansing, Mich.	Mt. Hope and Hagadorn	EB	2	0	0	1	1	4	72	78
7	Augusta	Greene and 7th	WB	3	1	0	1	1	36	48	43
8	Augusta	Telfair and 5th	EB	2	0	0	1	1	7	48	45
9	Augusta	Broad and 5th	WB	3	1	0	1	1	35	48	48
10	Atlanta	Blvd. and Angier	SB	2	0	0	1	1	45	48	61
11	Atlanta	Juniper and 10th	EB	3	2	—	1	1	34	48	48
12	Muttontown, N.Y.	Route 106 and Muttontown Road	NB	2	1	1	2	2	6	88	91
13	East Lansing, Mich.	Mt. Hope and Farmlane	EB	2	0	1	2	2	2	72	83
14	Sacramento	Watt and Whitney	NB	3	1	1	2	2	32	64	74
15	Brookline, Mass.	Beacon and Dean	WB	3	1	1	1	2	13	48	58
16	Brookline, Mass.	Kent and Aspinwall	EB	1	0	0	1	1	13	48	51
17	Lansing, Mich.	Michigan and Shepard	WB	2	0	1	1	1	4	40	50
18	Cambridge, Mass.	Main and Windsor	WB	1	0	0	1	1	13	48	48
19	Brookline, Mass.	Beacon and Dean	NB	1	0	0	1	2	33	48	48
20	San Francisco	Alemany and Geneva	WB	2	1	0	1	1	43	40	45
21	San Francisco	Alemany and Geneva	NB	3	1	0	1	1	43	40	61
22	San Francisco	Alemany and Geneva	SB	3	1	0	1	1	43	40	61
23	Sacramento	El Camino and Fulton	EB	3	1	1	2	2	23	56	66
24	Sacramento	Howe and Arden	WB	3	1	1	3	3	23	56	58
25	Augusta	Reynolds and 13th	WB	2	0	0	1	1	1	48	54
26	Atlanta	Ponce de Leon and Highland	EB	3	0	1	1	1	8	48	59
27	Boston	Beacon and Mass.	WB	3	2	—	3	2	13	48	56
28	Brookline, Mass.	Chestnut Hill and Dean	NB	1	0	0	3	2	44	48	58
29	Huntington, N.Y.	Oakwood and Pulaski	NB	2	0	0	1	1	6	64	72
30	Queens, N.Y.	Parsons and 77th	NB	1	0	0	2	1	39	48	50
31	Queens, N.Y.	Parsons and 78th	SB	1	0	0	2	1	31	48	53
32	Hicksville, N.Y.	Route 106 and West John (day)	SB	2	1	1	1	1	6	64	56
33	Hicksville, N.Y.	Route 106 and West John (night)	SB	2	1	1	1	1	6	64	59
34	Glen Head, N.Y.	Glen Cove and Glenhead Road	NB	2	1	1	2	3	6	88	88
35	Huntington, N.Y.	Oakwood and Pulaski	EB	1	0	0	1	1	2	80	78
36	Huntington, N.Y.	Park Avenue and Maplewood Road	SB	1	0	0	1	1	1	64	—[f]
37	Huntington, N.Y.	Park Avenue and Broadway	NB	1	0	0	1	1	1	64	—[f]
38	Huntington, N.Y.	Pulaski Road and Lake Road	SB	1	0	0	1	1	1	56	—[f]
39	Huntington, N.Y.	DeForest Road North and East Deer Park Avenue	WB	1	0	0	1	1	1	48	—[f]

Note: 1 km = 0.6 mile.

[a]NB = northbound, EB = eastbound, WB = westbound, and SB = southbound.
[b]0 = no, 1 = yes, and 2 = one way.
[c]0 = no, and 1 = yes.
[d]1 = fixed time, 2 = vehicle actuated, and 3 = pedestrian actuated.
[e]1 = 200 mm, 2 = 300 mm, and 3 = 300 mm, 200 mm, and 200 mm.
[f]Data not collected.

Figure 1. Signal configurations at data collection sites.

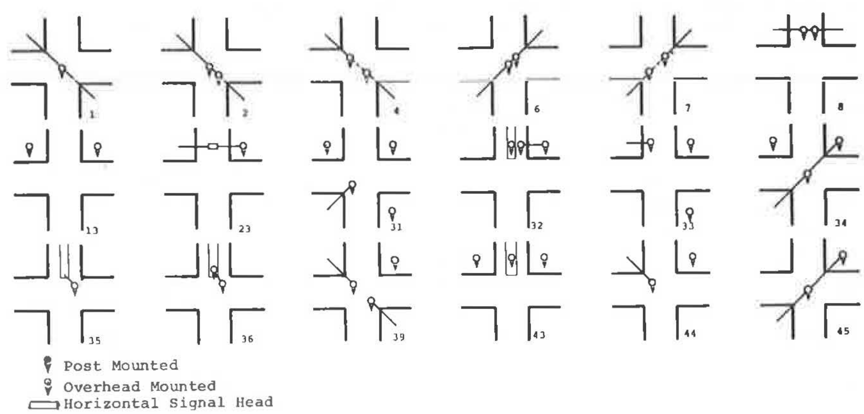
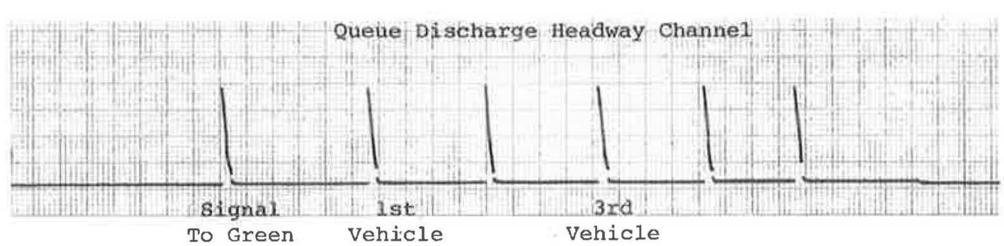


Figure 2. Sample recorder output showing queue discharge headway.



ANALYSIS OF QUEUE DISCHARGE HEADWAY DATA

The computed mean and standard deviation of the discharge headways for each queue position for each approach are shown in Table 2. The data were analyzed in general and then specifically for signal design configuration, signal lens size, and number of signal heads.

Overall Analysis

Queue discharge behavior has been studied by many investigators including Gerlough (2), Greenshields (3), Kell as reported by Gerlough (2), Berry and Gandhi (4), Capelle and Pinnell (5), and Carstens (6). Many researchers, however, have reported their results in average headway for the entire queue. The results of a number of studies that reported headways for individual queue positions are plotted in Figure 3.

Most of the previous studies support present results except for the first queue position. This lack of support is undoubtedly due to the definition of the measuring screen line used in the present study. Data on the important effect that this definition has on the results have been quoted by Berry and Gandhi (4). Since the present study is concerned with reactions to traffic signal configurations, a definition has been adopted that emphasizes the reaction time rather than the acceleration el-

Table 2. Overall analysis of queue discharge headways.

Location No.	Configuration No.	Position 1			Position 2			Position 3			Position 4			Position 5			Positions 6 to 8		
		No.	Mean	Std. Dev.	No.	Mean	Std. Dev.	No.	Mean	Std. Dev.	No.	Mean	Std. Dev.	No.	Mean	Std. Dev.	No.	Mean	Std. Dev.
1	13	28	2.26	0.55	28	2.49	0.76	23	2.70	0.64	22	2.64	0.87	15	2.64	0.83	22	2.58	0.89
2	13	31	2.32	0.79	31	2.90	0.81	29	2.48	0.51	24	2.22	0.52	15	2.22	0.77	18	2.17	0.80
3	13	29	3.36	0.97	29	3.17	0.78	28	2.72	0.64	17	2.60	0.40	10	2.22	0.78	4	2.05	0.55
4	13	28	3.67	1.56	28	3.09	0.83	28	2.71	0.52	16	2.51	0.68	7	2.59	0.56	1	2.00	0.00
5	4	36	2.30	0.88	35	3.00	0.68	30	2.40	0.55	23	2.20	0.58	17	2.29	0.69	20	2.19	0.63
6	4	33	2.31	0.69	33	2.78	0.60	22	2.27	0.31	11	2.79	0.63	4	1.85	0.39	2	2.45	0.35
7	36	29	2.73	0.92	29	3.48	0.73	23	2.53	0.63	18	2.52	0.67	13	2.33	0.55	4	2.30	0.73
8	7	30	2.87	1.26	29	3.04	0.79	20	2.65	0.60	16	2.27	0.67	2	1.85	0.21	0	0.00	0.00
9	35	30	2.85	1.29	29	3.40	0.67	20	3.08	0.62	7	3.04	0.87	5	2.26	0.24	3	2.03	0.49
10	45	30	2.82	0.95	30	3.09	0.66	28	2.89	0.75	23	2.53	0.62	14	2.38	0.88	10	2.41	0.60
11	34	31	2.58	1.02	29	3.08	0.85	20	2.69	0.73	12	2.65	0.62	2	2.50	0.71	0	0.00	0.00
12	6	41	1.80	0.66	41	2.79	0.66	24	2.23	0.51	14	1.95	0.52	7	1.76	0.38	7	1.63	0.72
13	2	30	2.19	0.70	30	2.79	0.56	30	2.16	0.57	24	2.13	0.43	19	1.98	0.38	19	2.07	0.76
14	32	30	3.31	1.33	29	3.47	0.90	28	2.63	0.91	22	2.47	0.45	11	2.32	0.67	4	1.93	0.46
15	13	27	2.61	1.17	27	2.70	0.66	27	2.51	0.54	20	2.57	1.26	10	2.23	0.45	4	2.83	1.09
16	13	37	2.64	0.79	32	2.53	0.43	18	2.57	0.81	9	3.21	1.09	4	2.23	0.17	2	2.70	0.28
17	4	33	2.55	0.96	32	2.75	0.50	31	2.45	0.59	28	2.22	0.37	18	2.34	0.53	14	2.49	0.65
18	13	30	2.14	0.69	20	2.81	0.58	11	2.56	0.55	7	2.64	0.82	4	2.00	0.36	2	1.50	0.71
19	33	20	2.13	0.98	20	2.88	0.69	20	2.25	0.58	16	2.08	0.46	12	1.99	0.46	21	2.20	0.66
20	43	31	2.78	1.44	31	3.25	0.58	31	2.62	0.64	21	2.22	0.55	15	2.21	0.68	9	2.01	0.35
21	43	29	2.97	1.16	29	3.29	0.76	29	2.30	0.32	22	2.52	0.89	14	2.05	0.30	8	2.54	0.58
22	43	31	3.33	1.22	31	3.05	0.81	31	2.44	0.51	22	2.66	0.54	12	2.22	0.67	7	2.27	0.56
23	23	30	3.07	1.31	30	2.87	0.74	30	2.40	0.75	21	2.24	0.55	11	2.57	0.81	2	1.55	0.35
24	23	29	2.94	0.87	29	3.22	0.78	29	2.36	0.72	20	2.20	0.61	13	2.12	0.56	8	2.06	0.50
25	1	28	2.24	0.71	23	3.42	0.68	17	3.21	1.06	11	3.05	0.65	7	2.51	0.61	2	2.05	0.21
26	8	30	2.11	0.90	30	2.79	1.39	28	2.39	0.64	26	2.30	0.62	14	2.19	0.64	13	2.19	0.50
27	13	27	2.09	0.78	27	2.66	0.68	25	2.30	0.71	24	2.27	0.60	22	2.25	0.82	34	2.08	0.55
28	44	35	2.28	0.87	35	2.83	0.80	32	2.42	0.81	30	2.18	0.47	20	2.25	0.57	19	1.89	0.47
29	6	30	2.15	0.73	19	3.05	0.52	13	2.79	0.65	9	2.16	0.30	2	2.00	0.99	1	1.50	0.00
30	39	30	2.82	1.43	19	2.93	0.58	11	2.45	0.45	7	2.39	0.43	4	1.93	0.30	1	2.10	0.00
31	31	30	2.54	1.03	20	3.30	0.58	13	2.79	0.70	7	2.73	0.66	0	0.00	0.00	0	0.00	0.00
32	6	30	4.02	1.04	18	3.41	0.97	13	2.84	0.62	5	2.44	0.50	3	2.87	0.46	0	0.00	0.00
34	6	30	2.96	1.14	21	3.10	0.98	17	2.76	0.65	9	2.02	0.45	4	2.23	0.53	4	1.85	0.42
35	2	30	2.72	1.01	19	3.62	0.93	14	2.34	0.52	8	2.05	0.48	6	2.33	0.37	3	2.30	0.90
36	1	33	2.57	1.31	23	3.04	0.55	19	2.57	0.59	10	2.23	0.70	4	2.25	0.24	2	2.30	0.71
37	1	31	2.23	0.61	27	2.99	0.79	20	2.29	0.56	9	2.22	0.47	5	1.60	0.20	5	2.22	0.60
38	1	31	2.25	1.15	27	2.85	0.50	20	2.42	0.51	14	2.18	0.36	9	1.83	0.43	10	1.96	0.46
39	1	30	2.04	0.88	30	2.63	0.52	27	2.37	0.50	25	2.15	0.54	23	1.99	0.41	59	1.96	0.55

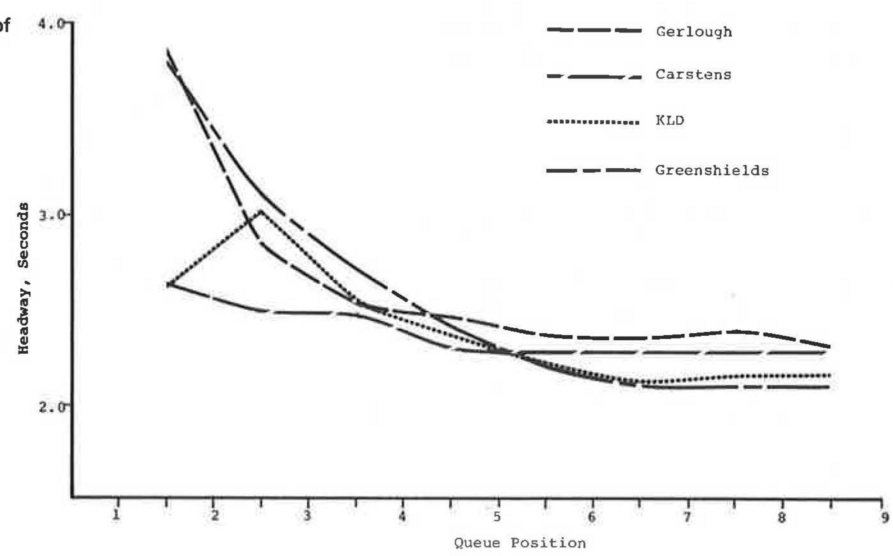


Figure 3. Comparison of various research results of queue discharge headway.

ement. Carstens (6), who adopted a similar definition, obtained similar results.

Aside from this difference in the first queue position, most data sets including the present show the same general trend: a decrease of discharge headway as queue position increases and then a leveling off to approximately 2.2 s by the fifth position.

Analysis by Configuration Class

Table 3 gives queue discharge headway data aggregated by signal configurations. Since only 2 of the 18 configurations were represented by five or more approaches, no separate analysis by individual signal design configurations was made. The major analysis was a comparison of four classes of signal design configuration: all post, single overhead, multiple overhead, and mixed.

Table 4 gives the data on queue discharge headway aggregated by configuration class. The same information is shown in Figure 4. These data reveal that there are differences in efficiency between the various configuration classes and the rank order of the configuration classes changes among queue positions.

For each queue position all possible duplicate comparisons were evaluated for statistical significance and yielded the following results at the $p = 0.05$ level.

1. At queue position 1 single overhead and multiple overhead configurations were better (i.e., lower value of start-up loss) than the all post or mixed configuration.
2. At queue position 2 the queue discharge headways were close together for all configuration classes (Figure 4). The only significant differences were between mixed and all post or multiple overhead.
3. At queue position 3 multiple overhead was better (i.e., queue discharge headway was lower) than the single overhead configuration. There were no other significant differences.
4. At queue position 4 the multiple overhead configurations were significantly better than at the other three. Also the mixed configurations were better than the all post configurations.
5. At queue position 5 the single overhead configurations were better than the all post or mixed configuration. There was, however, only a small sample available for this configuration.
6. At other queue positions data for all queue positions higher than five were merged for analysis because of the small number of data points and the fact that queue discharge headways tend to be constant beyond the fifth queue position. For these queue positions, single overhead configurations were significantly better than all post configurations. There were no other significant differences.

Analysis by Lens Size

Figure 5 shows the effect of lens size on queue discharge headway. The data set was partitioned according to size of the green signal lens in each configuration. The figure shows that the 300-mm (12-in) lens performs better for all queue positions. Statistical tests show that these differences are significant for the third and all subsequent queue positions. The hypothesis that the larger signal size is more likely to be used in the better performing overhead configurations was tested. For the third queue position a two-way analysis of variance showed that both configuration class and lens size were significant. The third position was selected for this test because it was the position at which significant differences first became apparent. Furthermore, for the same queue position, a comparison of 200-mm (8-in) and 300-mm (12-in) green lens sizes for all post configurations showed only that the larger lens performed significantly ($p = 0.01$) better than the smaller one.

Analysis by Number of Signal Heads

Figure 6 shows the apparent effect of the number of signal heads, without regard to configurations, on discharge headway. None of the three possibilities tested dominates, although the two-signal head combination appears to perform well.

The apparent good performance of the single overhead configuration can probably be explained by the fact that for both the two-head and three-head configuration groups more than half of each data set applied to all post configurations.

IMPACT ON DELAY AND CAPACITY

The separate analyses discussed in the previous section indicate that, except for lens size, no configuration-related factor can be considered better or worse at all queue positions. Also delay of a vehicle in a given queue position is not linearly related to the queue discharge headway for that position. If H_i is the discharge headway for the ith queue position, then the total delay (D_i) occurring to a vehicle in that position is given by

$$D_i = \sum_{j=1}^{j=i} H_j \tag{1}$$

The total delay (D_s) accrued by a queue of length k is then given by

$$D_{s(k)} = \sum_{j=1}^{j=k} D_j \tag{2}$$

or

$$D_{s(k)} = kH_1 + (k - 1)H_2 + \ldots + 2H_{k-1} + H_k \tag{3}$$

Table 5 gives the total delay, estimated by using this equation, for all vehicles in queue for some representative queue sizes. The units of delay in this table are expressed as vehicle•s/cycle. For a 60-s cycle, this is equivalent to vehicle•min/h.

Examination of this table shows that all post and mixed configurations accrue more delay for all four queue lengths. When the queue lengths are reached, the better performance of overhead configurations, both single and multiple, becomes more apparent. Improvement can also be expected with the use of the larger signal lens.

The computational method used to develop this table does not permit an assessment of significance that has a practical meaning because the variance for a linear combination of variables is the sum of the variances of the individual variables. For a queue of length k this represents $[k(k + 1)]/2$ items.

Although the differences in aggregate delay are not very striking, they indicate a definite improvement in approach capacity as the queue discharge headways decrease.

Table 6 gives computed capacities as a function of cycle length for two lens sizes and three basic configuration classes. Single overhead configurations were not included because of a shortage of data in the fourth to sixth queue position. These capacities were computed under the following conditions.

1. The maximum number of vehicles that can be serviced during a green interval are in queue at the beginning of the green signal. This condition means continu-

ous cycle failure or that both peak-hour factor and load factor are assumed to be equal to 1.0 as stated in the Highway Capacity Manual (HCM) (7).

2. The yellow interval is fully used for discharging vehicles.

3. The queue discharge headway is constant for all vehicles beyond the sixth queue position.

The capacity computation consists of discharging vehicles according to measured headways during the entire green-plus-yellow interval and multiplying the number discharged per cycle by the number of cycles in an hour. Because of the probabilistic nature of the individual headway values, fractional discharges per cycle were permitted. The algorithm used is as follows:

$$Q = \left\{[(C/2) - \sum_{i=1}^{6} H_i]/H_7 + 6\right\}(3600/C) \quad (4)$$

Table 3. Analysis of queue discharge headway by signal configuration.

Configuration No.	Number of Locations	Position 1			Position 2			Position 3			Position 4			Position 5			Positions 6 to 8		
		No.	Mean	Std. Dev.	No.	Mean	Std. Dev.	No.	Mean	Std. Dev.	No.	Mean	Std. Dev.	No.	Mean	Std. Dev.	No.	Mean	Std. Dev.
1	5	153	2.27	0.96	130	2.96	0.38	103	2.54	0.42	69	2.32	0.30	48	2.02	0.18	78	1.99	0.29
2	2	60	2.46	0.76	49	3.11	0.52	44	2.20	0.31	32	2.11	0.20	25	2.06	0.14	22	2.10	0.60
4	3	102	2.38	0.73	100	2.85	0.37	83	2.38	0.28	62	2.31	0.27	39	2.27	0.39	36	2.32	0.44
6	4	131	2.65	0.83	99	3.02	0.63	67	2.59	0.30	37	2.08	0.24	16	2.12	0.39	12	1.69	0.91
7	1	30	2.87	1.26	29	3.04	0.79	20	2.65	0.60	16	2.27	0.67	13	2.33	0.55	00	0.00	0.00
8	1	30	2.11	0.90	30	2.79	1.39	28	2.39	0.64	26	2.30	0.62	14	2.19	0.64	13	2.19	0.50
13	8	237	2.63	0.93	222	2.79	0.52	189	2.57	0.40	139	2.51	0.68	87	2.32	0.59	87	2.26	0.60
23	2	59	3.01	1.29	59	3.04	0.60	59	2.38	0.56	41	2.22	0.35	24	1.15	0.52	10	1.96	0.31
31	1	30	2.54	1.03	20	3.30	0.58	13	2.79	0.70	7	2.73	0.66	00	0.00	0.00	00	0.00	0.00
32	1	30	3.31	1.33	29	3.47	0.90	28	2.63	0.91	22	2.47	0.45	11	2.32	0.67	40	1.93	0.46
33	1	20	2.13	0.98	20	2.88	0.69	20	2.25	0.58	16	2.08	0.46	12	1.99	0.46	21	2.20	0.66
34	1	31	2.58	1.02	29	3.08	0.85	20	2.69	0.73	12	2.65	0.62	2	2.50	0.71	00	0.00	0.00
35	1	30	2.85	1.29	29	3.40	0.67	20	3.08	0.62	7	3.04	0.87	5	2.26	0.24	3	2.03	0.49
36	1	29	2.73	0.92	29	3.48	0.73	23	2.53	0.63	18	2.52	0.67	13	2.33	0.55	4	2.30	0.73
39	1	30	2.82	1.43	19	2.93	0.58	11	2.45	0.45	7	2.39	0.43	4	1.93	0.30	10	2.10	0.00
43	3	91	3.03	1.70	91	3.19	0.54	91	2.46	0.27	65	2.47	0.49	41	2.16	0.36	24	2.26	0.29
44	1	35	2.28	0.87	35	2.83	0.80	32	2.42	0.81	30	2.18	0.47	20	2.25	0.57	19	1.89	0.47
45	1	30	2.82	0.95	30	3.09	0.66	28	2.89	0.75	23	2.53	0.62	14	2.38	0.88	10	2.41	0.60

Table 4. Analysis of queue discharge headway by signal configuration classes.

Class	Number of Locations	Position 1			Position 2			Position 3			Position 4			Position 5			Positions 6 to 8		
		No.	Mean	Std. Dev.	No.	Mean	Std. Dev.	No.	Mean	Std. Dev.	No.	Mean	Std. Dev.	No.	Mean	Std. Dev.	No.	Mean	Std. Dev.
All posts	11	328	2.74	1.05	313	2.91	0.75	280	2.53	0.59	204	2.50	0.78	128	2.27	0.68	111	2.26	0.71
Mixed	10	294	2.73	1.12	270	3.12	0.77	234	2.53	0.75	176	2.36	0.57	100	2.27	0.64	69	2.10	0.58
Multiple overhead	11	353	2.51	1.07	307	2.96	0.93	242	2.43	0.59	173	2.22	0.54	96	2.17	0.56	83	2.15	0.67
Single overhead	6	183	2.37	1.05	159	3.02	0.68	123	2.63	0.72	76	2.39	0.67	53	2.04	0.47	81	1.99	0.53
All	—	1158	2.61	1.11	1049	3.00	0.77	879	2.52	0.66	629	2.37	0.66	377	2.21	0.62	344	2.14	0.64

Figure 4. Effect of configuration on queue discharge headway.

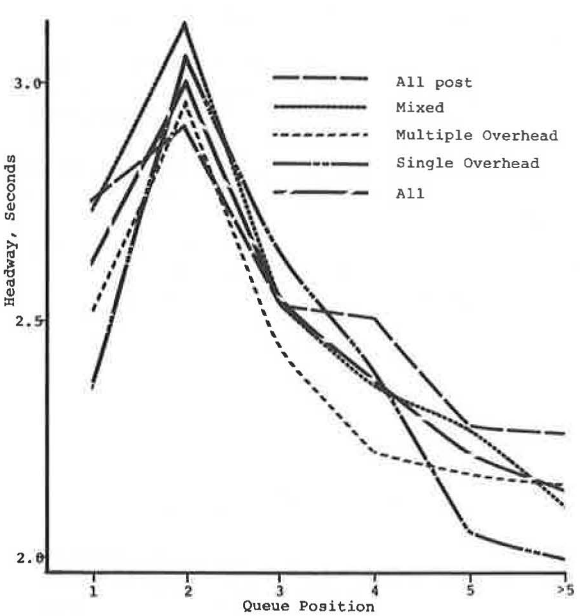
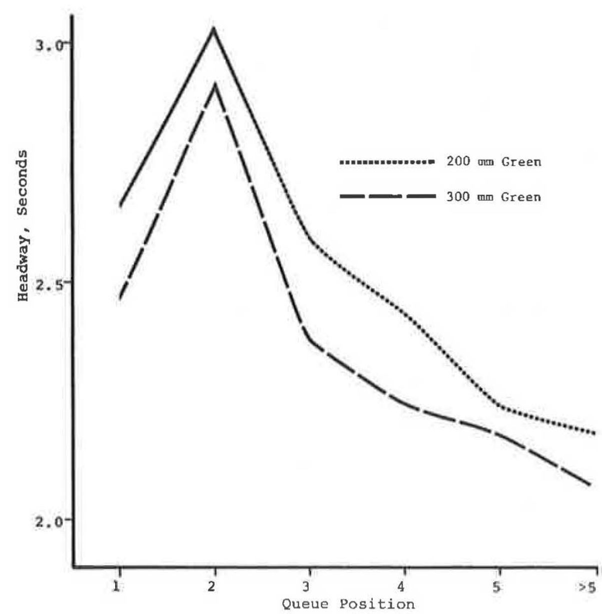


Figure 5. Effect of lens size on queue discharge headway.



where

Q = capacity in vehicles per hour,
C = cycle lengths in seconds (50 percent split assumed),
H_i = discharge headway of ith vehicle ($i < 7$), and
H_7 = average discharge headway of seventh and following vehicles.

Figure 6. Effect of number of signal heads on queue discharge headway.

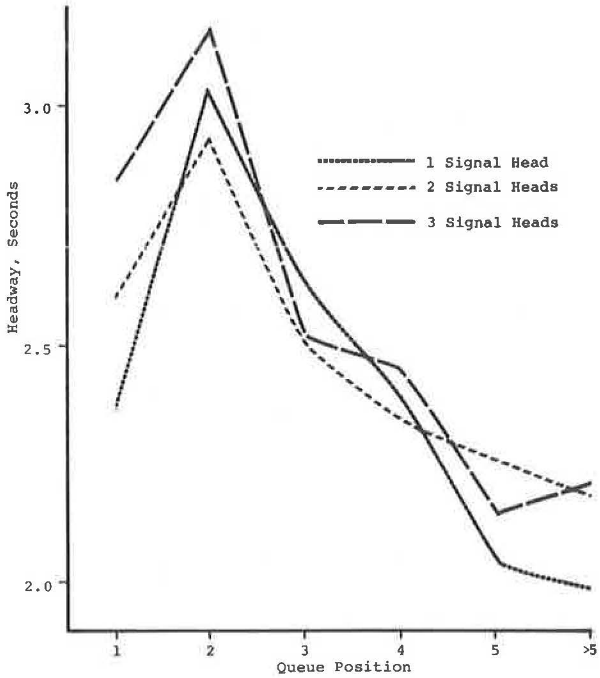


Table 5. Estimated total delay in queue.

Signal	Delay (vehicles/s/cycle) by Queue Length			
	2 Vehicles	4 Vehicles	6 Vehicles	8 Vehicles
All post	8.4	27.2	55.4	92.6
Mixed	8.6	27.7	55.8	92.3
Multiple overhead	8.0	26.0	52.7	88.1
Single overhead	7.8	26.2	53.2	88.1
30.5-cm green lens	7.8	25.6	52.0	86.7
20.3-cm green lens	8.4	27.3	55.4	92.2
Two heads	8.1	26.5	53.9	90.1
Three heads	8.8	28.3	56.8	94.1
All approaches	8.2	26.9	54.4	90.5

Note: 1 cm = 0.39 in.

Table 6. Signal configuration and approach capacity.

Signal	Capacity (vehicles/h/lane) by Cycle Length		
	45 s	60 s	90 s
All post	742	748	755
Mixed	774	792	810
Multiple overhead	805	817	829
30.5-cm green lens[a]	809	824	839
20.3-cm green lens[a]	770	780	790
All approaches[a]	786	799	812

Note: 1 cm = 0.39 in.
[a] Data for single-overhead configurations are included.

The data in Table 6 show that the difference in computed capacity between all post and multiple overhead configurations averages 9 percent for the three cycle lengths or nearly 70 vehicles/h. The computed values of 1442 to 1658, averaging 1572 vehicles/h of green per lane, compare well with the capacities for signalized approaches computed according to the procedures of chapter 6 of the HCM. For example, the computed capacity of a signalized approach where there are two 3.4-m (11-ft) lanes, parking is permitted, and turns and trucks are prohibited is 1566 vehicles/h of green per lane.

ACKNOWLEDGMENT

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The opinions and conclusions expressed or implied in the report are ours and are not necessarily those of the Transportation Research Board, the National Academy of Sciences, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, or of the individual states participating in the National Cooperative Highway Research Program.

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Discussion

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The conclusions given in this paper are based primarily on analysis of differences in queue discharge behavior in relation to differences in traffic signal configurations;

queue discharge measurements were made at 38 different signal approaches in five states.

My discussion deals primarily with the selection of the screen lines where queue discharge headways are measured and with differences that can be expected in headways when different screen line definitions are used. The authors used two different screen-line definitions in collecting their data. They state that data on queue discharge behavior for each cycle were recorded as each vehicle passes the stop line or a screen line, which was established as the location of the front wheels of the car first in queue.

Figure 7 shows use of a screen line located at the position of the stopped front wheels of the first vehicle in queue. In alternative IA, the first vehicle would be considered to have started when it begins motion. The elapsed time from the beginning of the green interval would include reaction time, but no acceleration time. In alternative IB, vehicle 1 would be considered discharged when its rear wheels crossed the screen line as established by the stopped front wheels. The start-up time thus would include reaction time and the time to accelerate a distance equivalent to the wheelbase length of vehicle 1. Alternative IB apparently was used by the authors for a portion of their cycles for some or all of their signal configuration groupings.

In alternative II the stop line is used as the screen line. The two versions relate to whether the front or the rear wheels were used to identify when the vehicle passed the stop line. The authors used the stop line for some of the observations (presumable alternative IIA). In alternative III, the crosswalk line is the screen line for measuring queue discharge behavior. The entry to the intersection is the screen line in alternative IV.

In 1973, Kittelson (8) investigated the effect of two screen lines on queue discharge headways. Time-lapse photography was used at 5 frames/s at a single-lane approach adjacent to the Evanston campus of Northwestern University. His films have data for analyzing effects of five screen-line definitions on starting delay for the first vehicle and on headways for subsequent vehicles.

Figure 8 shows average starting delays for the first vehicle and average headways for the next three vehicles for five screen-line definitions; data from the same eight queues of vehicles (eight cycles) were used (30-cycle averages for start-up delay for vehicle 1 are as follows for three screen lines: IB, 2.71 s; IIIB, 3.30 s; and IV, 4.25 s). The choice of a screen-line definition affects headways for both queue position 1 (starting delay) and queue position 2.

Figure 7. Alternative screen lines for measuring queue discharge headings at signals.

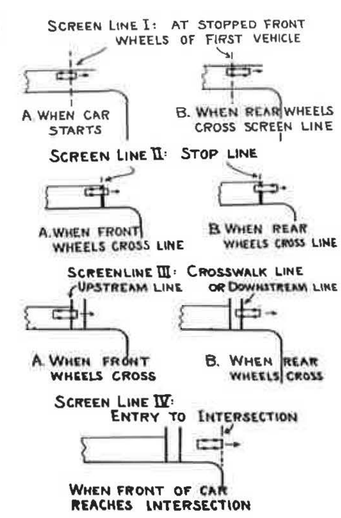
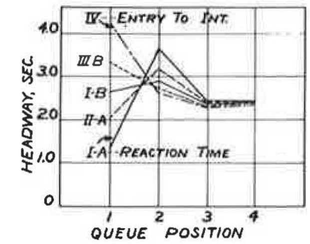


Figure 8. Start-up delays and headways for five screen-line types.



King and Wilkinson use both screen-line definitions IIA and IB and give no indication of the proportion of cycles for which each screen-line type is used. Reexamination of their data would be desirable to determine the proportion of cycles in each grouping of signal configurations that used each of the two screen-line types.

Other location factors that affect the length of the start-up time for the first queued vehicle for different screen-line definitions include

1. The distances between the stop line, the crosswalk lines, and the point of intersection entry;
2. The extent to which drivers tend to stop behind the stop and crosswalk lines when stopping;
3. The extent to which the side-street yellow signal is visible to the drivers; and
4. Whether a yellow signal is displayed after the red and just prior to the green, as in some European countries.

Schwarz (9) studied starting delays in 1961 at seven intersections in Chicago before and after elimination of a "get ready to go" yellow varying from 1.7 to 2.6 s. Using screen-line definition IIIB, he found that starting delays with the advance yellow averaged 1.20 s lower (2.97 versus 4.17) than with the red-green sequence. The differences were significantly different. Distances from stop lines to his crosswalk screen lines varied from 5.6 to 11.5 m (18.4 to 37.8 ft).

George and Heroy (10) studied starting reaction times at five signalized intersection approaches and found average start-up times per intersection varying from 1.5 to 2.0 s. His criterion, corresponding to screen-line definition IA, probably would have been preferable for an analysis of differences in response to different signal configurations. Greenshields (3) used definition IV for his headway studies, but he also reported reaction times for screen-line definition IA.

For capacity analysis in which measuring use of the yellow for loaded cycles is also desirable (4), I would recommend making queue discharge measurements at the entry to the intersection, corresponding to screen-line definition IV.

Authors' Closure

Berry's discussion of the influence of screen-line selection on the numerical values of queue discharge headway is a valuable and necessary contribution to the subject. He correctly points out that this selection is important in the relationship of acceleration and reaction time to the correct computation of queue discharge headway measurements.

First, we would like to clarify the exact measurement technique used to obtain the data presented in our paper. In keeping with the overall purpose of the research, we attempted to define a screen line that would emphasize the reaction time element. Furthermore, since manual

Figure 9. Comparison of queue discharge headway data.

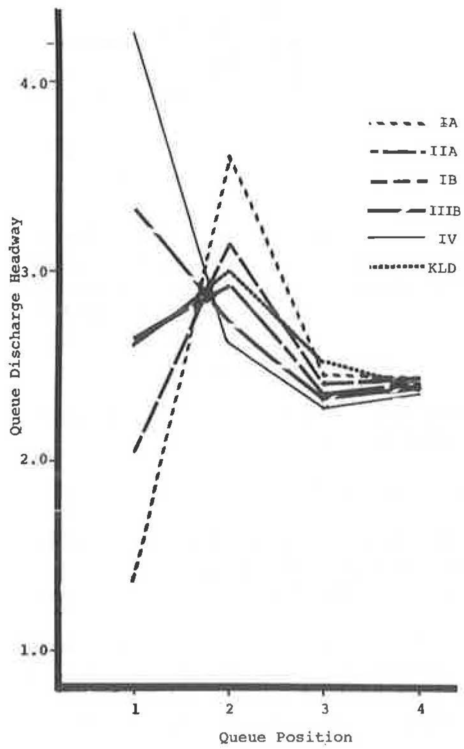
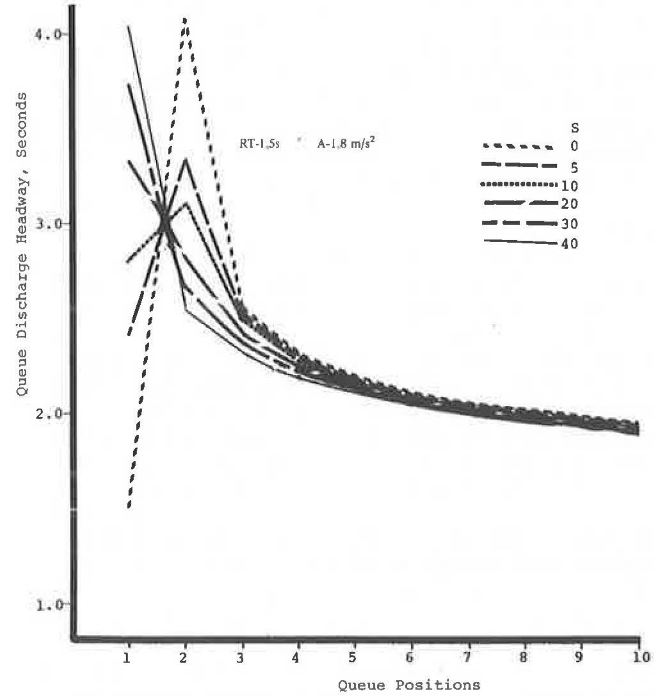


Figure 10. Relation of distance of first vehicle to screen line and queue discharge headway.



data inputs were used, a screen line visible in the field had to be selected. We therefore chose the stop line as the primary screen line on the assumption that most vehicles would be stopped with their front wheels only a short distance behind that line. However, an appreciable number of vehicles came to a stop straddling the stop line. For those vehicles, the stop line was retained as the screen line, but the passage of the rear wheels over that line was recorded. All measurements were consistent for any one queue. Either the front or rear wheels were used depending on the stopped position of the first vehicle. The entire data base is thus a random mixture of those two types; no records were kept of individual queues. The data base thus represents a mix of Berry's definitions IB and IIA, as he points out and as shown in Figure 9, in which our data are superimposed on Berry's data shown in Figure 8.

The empirical results presented by Berry in Figure 7 can be supported by theoretical analysis. Starting with the basic laws of motion, one can show that the discharge time of the nth vehicle (T_n) in queue can be computed by equation 5 if the simplifying assumptions of uniform space headways in queue, uniform acceleration, and constant reaction time are made.

$$T_n = \sqrt{2\{[S + H(n-1)]a\}} + RTS + (n-1)\,RTV \qquad (5)$$

where

RTS = signal perception-reaction time in seconds,
RTV = vehicle motion perception-reaction time in seconds,
H = space headway in queue in meters,
S = distance from first vehicle to screen line in meters, and
a = acceleration in meters per second squared.

The queue discharge headway (ΔT_n) represents the difference in queue discharge times of the nth and (n - 1)th vehicle and is given by

$$\Delta T_n = RTV + \sqrt{(2/a)}\left[\sqrt{S + H(n-1)} - \sqrt{S + H(n-2)}\right] \qquad n > 1$$

$$\Delta T_n = RTS + \sqrt{2S/a} \qquad n = 1 \qquad (6)$$

Figure 10 shows this relationship for the following representative parameters: RTV = RTS = 1.5 s, a = 1.8 m/s² (6 ft/s²), and H = 6.1 m (20 ft). The generic resemblance to Berry's data is apparent.

Berger-Robertson Method for Measuring Intersection Delay

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Vehicle delay is generally viewed by traffic engineers as a tangible measure of intersection performance, but not an easily obtained measure. Two major problems are encountered in the measurement of delay. The first problem is fundamental. Any measurement technique must be based on a conceptual model of the phenomenon being measured and most models that have been developed have probabilistic properties. However, theoreticians have generally limited their models to deal with circumstances of constant average volume levels that are below capacity. This technique makes the model generally inapplicable to peak-hour conditions in which traffic volume levels are not constant, do not follow any mathematical distribution, and often exceed capacity.

The second problem is that conventional methods of field measurement of vehicle delay generally require the microscopic observation of the traffic stream and recording of every vehicle involved. The considerable expense of these methods has generally limited their application to infrequent usage. The result is a lack of basic field data from which vehicle delay parameters can be developed.

BERGER-ROBERTSON METHOD

The Berger-Robertson method is a manual procedure that is based on several established mathematical and traffic engineering relationships. First, an estimate of a continuous function can be approximated by a linear fit over any sufficiently small interval. Second, where a linear equation is adequate, the center (or mean value) on that line represents the best least square estimate of the values contained in that region. Finally, when computing stopped delay, we assume that the earlier arriving vehicles in any one lane are released from the queue first. These three relations suggest a method for the manual collection of stop-time delay that should be bias-free relative to the saturation levels of the intersection and independent of the signal cycle length.

The procedure is to divide the total time period of interest (e.g., cycle length) into a sufficiently small number of equal intervals (e.g., 5s). Then the vehicles that stop in each interval are tallied separately, and the midpoint of the interval is assumed to represent the average arrivals of the vehicles in the interval. The number of previously stopped vehicles departing (clearing the intersection) is also tallied by interval. Again the departure of these vehicles is assumed to be randomly distributed in the interval.

A clipboard and tally sheet are required. The tally sheet consists of two rows (one for stopping and one for starting) divided into equal blocks representing the number of intervals in the sampling period. Having an interval timer with an auditory tone is helpful for signaling the onset of each interval.

The results from the above procedure have been compared with those obtained by using time-lapse photography (where frame counts per vehicle were recorded). The time-lapse film used for this comparison was taken during a recent project in the District of Columbia (1). The southbound approach on Wisconsin Avenue at the intersection of Western Avenue was used. The approach consists of three lanes (one left turn only and two through lanes). This approach carried 12 322 vehicles between 7 a.m. and 6 p.m. on the day the film was taken. The method was applied to 10 periods equally distributed through the 11-h data collection day. The results of the comparison appear in Table 1.

Two comparisons are provided in Table 1. The first represents the results of applying the Berger-Robertson method to the values obtained from the film scoring. This analytic use of the method provides a good indication of the extent to which real data meet the assumptions on which the method is based. The second comparison entailed viewing the film and manually coding the vehicles in real time. This application demonstrates the feasibility of real-time use of the method. The data obtained by manually coding the film included the component of human error and therefore provide some idea of the "real world" reliability of the method.

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Table 1. Comparison of intersection delay generated via time-lapse and Berger-Robertson method.

Signal Cycle	Left Lane Delay (vehicle·s) Time-Lapse	Left Berger-Robertson Analytic	Left Berger-Robertson Manual	Center Lane Delay (vehicle·s) Time-Lapse	Center Berger-Robertson Analytic	Center Berger-Robertson Manual	Right Lane Delay (vehicle·s) Time-Lapse	Right Berger-Robertson Analytic	Right Berger-Robertson Manual
1	27.5	25	25	40	40	40	46	45	45
2	58	50	65	185	185	165	0	0	0
3	29.5	30	30	83	95	80	41	35	40
4	2.5	5	5	87	85	90	38	35	35
5	80.5	75	70	79.5	75	70	15.5	15	15
6	412	405	405	86.5	80	95	52	50	55
7	186	185	185	144	140	135	66	65	55
8	120.5	120	120	60	60	55	64	60	55
9	439.5	440	445	120.5	115	115	21	20	20
10	592	580	590	283	280	300	138	140	155

Table 2. Statistical analysis of data in Table 1.

Lane	Method	Mean	Standard Deviation	Correlation Coefficient With Time-Lapse
Left	Time-lapse	194.80	209.36	-
	Berger-Robertson			
	Analytic	191.50	206.99	0.999
	Manual	194.00	208.99	0.999
Center	Time-lapse	116.85	71.87	-
	Berger-Robertson			
	Analytic	115.50	71.12	0.997
	Manual	114.50	75.00	0.991
Right	Time-lapse	48.15	37.92	-
	Berger-Robertson			
	Analytic	46.50	38.59	0.998
	Manual	47.50	42.18	0.998

CONCLUSIONS

Based on the data in Table 1, we can conclude that our method is theoretically sound and operationally practical. The theoretical soundness of the model is attested to by its analytic correspondence with the time-lapse data (Table 2). The means and standard deviations are nearly identical, and the correlation between the two procedures are uniformly high. The operational practicality of the method is reflected in a similarly consistent relationship between the time-lapse film data and the manual application of the method.

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Weighing in Motion in California

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Since the California Department of Highway Patrol commenced enforcing vehicle weight laws, the primary method of detecting overweight vehicles has been to stop the vehicles and weigh them on platform scales. To improve efficiency of vehicle weight enforcement and highway operation and to minimize delay and inconvenience to the trucking industry, the California Transportation Laboratory in cooperation with the Federal Highway Administration has designed and placed into operation a trial high-speed truck screening system. This installation is located on westbound Interstate 80 at the Cordelia weighing facility, about 80 km (50 miles) west of Sacramento. Using strain-gauge, load-cell weighing bridges, the system is capable of weighing trucks at speeds up to 56.3 km/h (35 mph). Initial tests indicate average errors of 2 or 3 percent and no more than 10 percent.

The high-speed scales are located approximately 243.8 m (800 ft) upstream from a 3.05 by 3.66-m (10 by 12-ft) platform scale, static weighing station at Cordelia. Maximum truck volume is approximately 2100 vehicles/d in August and decreases to 1100 vehicles/d in December.

The truck screening project has been in operation intermittently since November 1974. A statistical experiment was designed and performed in May 1975 with a loaded five-axle truck. Unfortunately a scale failed during the test and the results were not accurate. Six of the scales failed between November 1974 and June 1975. These were returned to the manufacturer, repaired, and reinstalled.

DESCRIPTION OF TRUCK SCREENING INSTALLATION

Power is furnished to the electronic scale chassis from 12 separate strain gauges so that scale calibration is easy. The force exerted by the tire on the transducer assembly produces an output signal voltage proportional to that force; each 44.5-kN (10 000-lbf) increment produces a 10-mV signal. The excitation voltage and the output signal voltage are brought to each transducer in 1.27-cm (0.5-in) diameter copper tubing that also supplies a protective envelope of dry nitrogen gas at a slight positive pressure of 13.8 kPa (2 lbf/in^2). The nitrogen gas is used to impede moisture from entering the strain-gauge load cells.

The output signal voltage (proportional to the vertical force) is connected to the multiplexer-analog to digital converter (DC). This device sequentially scans the transducers at a rate of 13 900 samples/s, amplifies the signal to an acceptable level, and then converts the magnitude of the DC signal to a corresponding binary code acceptable for transfer to the computer.

In the computer the vertical forces exerted by the wheel on the transducer assembly are averaged over the four successive samples that have values higher than a preselected cutoff value and that have the least deviation from their mean. This average force is then added to the average force obtained for the wheel on the other five scales as that wheel goes over each scale. This sum is then divided by six to obtain an overall average weight for that wheel. This average force is added to the average force obtained for the wheel on the other end of the axle to obtain an average axle weight. This process is repeated for each axle of the vehicle as it crosses each of the six pairs of transducers.

During the time the truck is dynamically weighed, the space between axles is also determined to the nearest 30.5 cm (1 ft). The axle weight and spacing are compared with those set forth in the vehicle code. If a violation is detected, the system prints the type of violation on the teletype and by overhead signals directs the truck driver to the platform scales to have the truck reweighed. If the truck is determined to be in violation of the weight laws, the driver is cited and must correct the violation before he can move the vehicle from the weighing station. Vehicle speed is also calculated, and, if a change in speed of more than 25 percent is detected, the truck driver is directed to the platform scales to have the truck weighed. If the truck is not in violation, the driver is directed by overhead signals to use a bypass

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lane to return to the freeway without stopping at the platform scales.

Violations are indicated to the weighmaster by a small display mounted in front of him. The weighmaster has control of the overhead signals and can direct any vehicle to come to the platform scale. This procedure permits him to determine whether a vehicle has some obvious safety deficiency and whether it needs a mechanical inspection.

CONCLUSIONS AND RECOMMENDATIONS

1. Weighing in motion on a day-to-day basis has proved to be practical.
2. Typical errors between the motion and static vehicle weights are 2 to 3 percent although some errors exceed 10 percent.
3. Additional work is required to improve the reliability of equipment.
4. Further developmental work is recommended to provide more reliable operational weighing transducers.

In the future, similar installations at major weighing facilities in California will serve to speed truck traffic and provide greater safety by reducing queues of trucks backed up onto the freeway waiting to be weighed.

Nonlinear Truck Factor for Two-Lane Highways

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A microscopic simulation model for traffic flows on two-lane, two-way highways was developed to include all important factors known to affect these flows. This simulation provided results in agreement with field data and was applied to flows in level terrain, in rolling terrain, and on sustained grades. Results from the model indicated that the truck factor, currently of linear form, should be nonlinear. A nonlinear form was derived and successfully applied to summarize results for a variety of terrains and vehicle populations. This paper presents a brief description of the simulation, the evidence for a nonlinear truck factor, and the derivation and testing of the nonlinear factor.

The Highway Capacity Manual (HCM) (1) presents methods for estimating the speeds and service on two-lane, two-way highways. The methods and numerics are based on data collected in the 1950s with revisions to account for the general increase in speeds prior to the 88.5-km/h (55-mph) national speed limit. The changes in vehicle speeds and populations since the 1950s raise questions regarding the adequacy of the methods and numerics in the HCM. This paper presents results from a National Cooperative Highway Research Program (NCHRP) project that contained several tasks designed to update the information on the vehicle population and to improve the methods for estimating speed and service on two-lane highways (2).

METHODS EMPLOYED

The characteristics of two-lane flows were evaluated by using a microscopic simulation model. The model was developed and adjusted by using data from the literature and data collected by St. John and Kobett (2). The latter extended the scope of information on passing behavior and provided samples of overall travel speeds on a test section with limited passing opportunities on rolling terrain.

The vehicle characteristics and vehicle populations used in the simulation model were based on field data and data from the literature. A subcontractor performed acceleration tests on a few recreational vehicles and combinations. Acceleration and speed performance data on passenger cars, trucks, and recreation vehicles were obtained from the literature. Analytical expressions were developed to relate acceleration capability to speed and local grade for trucks, passenger cars and light pickup trucks, motor homes, and other recreational vehicles and combinations. Thirteen vehicle types were used in the simulation model: three passenger cars, three trucks, and seven recreational vehicles and combinations.

The simulation model incorporates all known parameters that influence two-lane, two-way traffic flows. The parameters include

1. Acceleration and speed capability limits for each type of vehicle including the effect of the local grade;
2. Driver preferences that can restrain the use of performance capability in acceleration and speed maintenance;
3. Overtaking and following characteristics that provide realistic representation over the full range of conditions from high speeds to congestion;
4. Acceptance (and rejection) of passing opportunities based on distance to the next oncoming vehicle if it is in sight, passing sight distance, speed of impeding vehicle, location in impeded platoon, distance to end of passing zone if it is in sight, and presence or absence of horizontal curvature within the range of passing sight distance;
5. Vehicle lengths treated explicitly in overtaking, following, and in passing maneuvers;
6. Passing maneuvers subject to the constraints of vehicle acceleration and speed performance and also to the restraints that field data indicate are used by drivers;
7. Passing sight distance as a separate variable in each direction and local magnitudes consistent with alignment and with passing and no-passing zones; and
8. Multiple passes, i.e., one or more vehicles passing more than one impeding vehicle or more than one vehicle passing an impeding vehicle.

In addition, the following assumptions are made:

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Figure 1. Mean speeds of passenger cars on 0 percent grades based on vehicle population of 100 percent passenger cars.

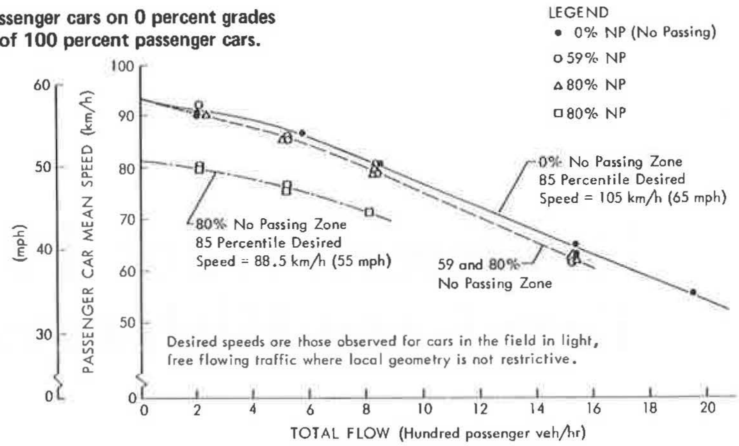
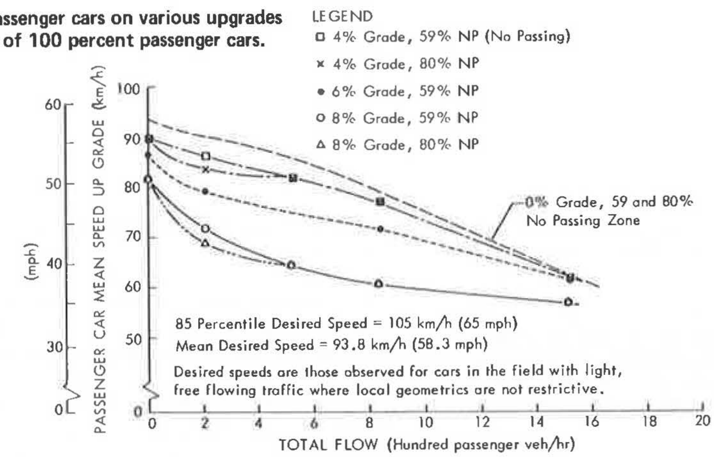


Figure 2. Mean speeds of passenger cars on various upgrades based on vehicle population of 100 percent passenger cars.



1. Vehicles in the model slow to negotiate horizontal curves that have combinations of curvature and superelevation requiring speed reduction;
2. Flying passes (where the impeded vehicle still has a speed advantage) are permitted, but before the pass decision, closure speeds are constrained by overtaking characteristics;
3. Passing maneuvers that become infeasible are aborted if the passer is not already committed to complete the pass (restraints on the use of vehicle performance are abandoned by the passer committed to pass when the maneuver becomes infeasible); and
4. Trucks use crawl speeds to descend sustained grades of 4 percent and steeper.

Results from the model combine to produce speed versus flow rate curves similar to those displayed in the HCM (1). When vehicle performance characteristics appropriate for the data collection period are used, the model also provides pass frequencies in agreement with data collected by Normann (3). When the features of the data collection site and observed vehicle population are used, the model also produces a distribution of passenger car speeds in close agreement with data collected in rolling terrain. When the overall speed data were collected in the field, the traffic flow contained a large percentage of trucks and a small percentage of recreational vehicles. However, during data collection only small changes occurred in the truck and recreational vehicle percentages. Consequently, the model has not been validated for its sensitivity to variations in the percentage of trucks.

BASIS FOR QUANTIFYING TRUCK FACTORS

A truck factor (F_T) is conventionally used to adjust the flow of mixed vehicles at a rate of Q vehicles/h to the equivalent flow rate of passenger cars only (Q_E).

$$Q_E = Q/F_T \qquad (1)$$

This relationship and application are retained here; as shown later the nonlinearity arises in the functional from of the truck factor.

The flow rate (Q) may consist of a mixture of passenger cars, recreational vehicles, and trucks. Q is equivalent to a Q_E of 100 percent passenger cars in only one respect. In the present case mean speed of passenger cars has been chosen as the measure for equivalence; other aspects of the flows are not necessarily similar.

If mean speed of passenger cars is the measure for equivalence, knowing and using these mean speeds are necessary for traffic flows with 100 percent passenger cars. Figure 1, based on simulation results, shows how the mean speeds vary in relation to highway properties and total flow rate on level terrain. The total flow is two-way, and the depicted results are obtained from the simulation model with nearly balanced flows.

Figure 2, based on simulation results, shows mean speeds of passenger cars on sustained grades of 0, 4, 6, and 8 percent. All results are for highways on which the mean spot speed of passenger cars is 93.80 km/h (58.28 mph) in very light traffic at locations with good geometrics and essentially 0 percent grade. (The 85th percentile speed is 105 km/h or 65 mph.) The results are for balanced flows; however, the speeds for the 4 to 8 percent grades are those in the upgrade direction. (The downgrade traffic is modeled just as accurately.) Based on 100 percent passenger vehicles, the downgrade mean speeds are slightly, but not significantly, higher than the 0 percent grade values.

The speed-flow rate relations in Figure 2 are the base for quantifying the equivalences described in this paper. The curves provide the necessary relations between mean speeds of passenger cars and flows of 100 percent passenger cars. The 0 percent grade curve is used for 0 percent grades, rolling terrain, downgrades, and 2 percent grades. The 4, 6, and 8 percent upgrade curves are used for sustained grades of those magnitudes. Application in this paper is restricted to the highway speed indicated, to essentially balanced flows, and to highway sections with 46 to 80 percent no-passing zones. (The extension to 46 percent no-passing was indicated by favorable experience with this value when the curves in Figure 2 and figures that follow were used.)

The following is an example of equivalent flows: Passenger cars in a mixed flow over rolling terrain have an overall mean speed of 76.8 km/h (47.7 mph), and the nearly balanced mixed flow rate is 600 vehicles/h. Since the equivalent flow rate of passenger cars has the same overall mean speed, the equivalent flow rate is read in Figure 2 on the 0 percent grade curve at 76.8 km/h as 925 passenger cars/h.

The speeds in Figure 2 may appear low for the 4, 6, and 8 percent grade. All vehicles in the model, including the population of passenger vehicles, have realistic acceleration and speed capabilities. Also, our analysis of data supplied by Werner (4) indicates that the drivers of passenger cars, light pickup trucks, and recreational vehicles do not use all the available vehicle power for extended periods. Consequently, the combination of performance characteristics and driver restraint does have a significant effect on passenger car flows on sustained grades of 4 percent and steeper. St. John, in presenting this topic in detail (2), shows that the speed data collected by Williston (5) are explained by the combined effects of performance limits and driver restraint. (In contrast, all of the available power in intercity transport trucks is used for extended periods.)

EVIDENCE FOR A NONLINEAR TRUCK FACTOR

Results from the simulation model supply strong evidence that the truck factor should have a nonlinear form. Figure 3 shows equivalents calculated from the linear form of the truck factor for three truck types, a low-performance camper, and a low-performance travel trailer combination. The equivalents are based on model results from simulation runs in which a single type of impeding vehicle is present. The equivalents are plotted against the travel speed of the impeding vehicle. The relation between the equivalent and the speed of the impeding vehicle has the general form shown in the HCM. However, there are two distinct curves. One curve connects points from model results in which there are 8 to 9 percent of one of the impeding vehicle types; the second curve connects points where there are 18 to 21 percent of the impeding vehicle type. These results indicate a type of nonlinearity. For example, if 65.8-km/h (40.9-mph) vehicles replace 10 percent of a passenger car flow, the 65.8-km/h vehicles are each equivalent to 15 passenger cars. However, if 20 percent of the passenger cars are replaced, each of the slow vehicles would be equivalent to only 8.5 passenger cars. From an incremental standpoint, the second 10 percent are less disruptive to the flow than the first 10 percent. (The first 10 percent have already depressed speeds.) The equations currently employed to obtain truck factors with equivalents assume a linearity that is inconsistent with the simulation results.

Figure 3 and the above example deal with instances in which different fractions of the same vehicle type were compared for effect. A similar and consistent nonlinearity is found for cases in which two or more types of impeding vehicles are involved. The effect of the mixture is not predicted correctly from the effects of the individual types when they are combined by using the current linear expression for the truck factor.

DERIVATION OF A NONLINEAR TRUCK FACTOR

An alternative version of the truck factor equation that may be derived to establish a relation that depends exclusively on the speed of the low-performance vehicle is applicable to a range of truck (or recreational vehicles) percentages, and correctly combines and predicts the influence of a mix of low-performance vehicle types.

We retain the concept expressed in equation 1. The factor $1/F_T$ is written

$$1/F_T = 1 + (P_T/100)(E_T - 1) \tag{2}$$

where

P_T = percentage of trucks, and
E_T = equivalents of trucks.

However, the application of the above form is restricted to adding small increments of percentages of trucks as shown by the following equation.

$$1/F_T = 1 + (\delta P_T/100)(\nu - 1) \tag{3}$$

where

δP_T = small increment of total percentage of trucks, and
ν = form of equivalence associated with the incrementally added trucks.

When the first increment of passenger cars is replaced by the increment of trucks, the equivalent flow is Q_{E1}.

$$Q_{E1} = Q[1 + (\delta P_T/100)(\nu - 1)] \tag{4}$$

Now, before the second increment of cars is replaced by trucks the traffic has characteristics associated with flow rate Q_{E1}, which is larger than Q. Consequently, the effective percentage of the second increment is $(Q/Q_{E1})\delta P_T$. After the second increment is added the equivalent flow is Q_{E2}.

$$Q_{E2} = Q[1 + (\delta P_T/100)(\nu - 1)(1 + Q/Q_{E1})] \tag{5}$$

We now recognize that the incremental change in Q_E is $Q_{E2} - Q_{E1}$, which can be written as

$$\delta Q_E = Q[(\delta P_T/100)(\nu - 1)(Q/Q_{E1})] \tag{6}$$

In the limit the incrementals δQ_E and δP_T become differentials, and equation 6 becomes a differential equation that integrates to

$$(Q_E/Q)^2 = 2(P_T/100)(\nu - 1) + \text{constant} \tag{7}$$

However, when $P_T = 0$, $(Q_E/Q) = 1.0$ so that the equation has the form

Figure 3. Vehicle equivalents depending on speed and percentage of impeding vehicle.

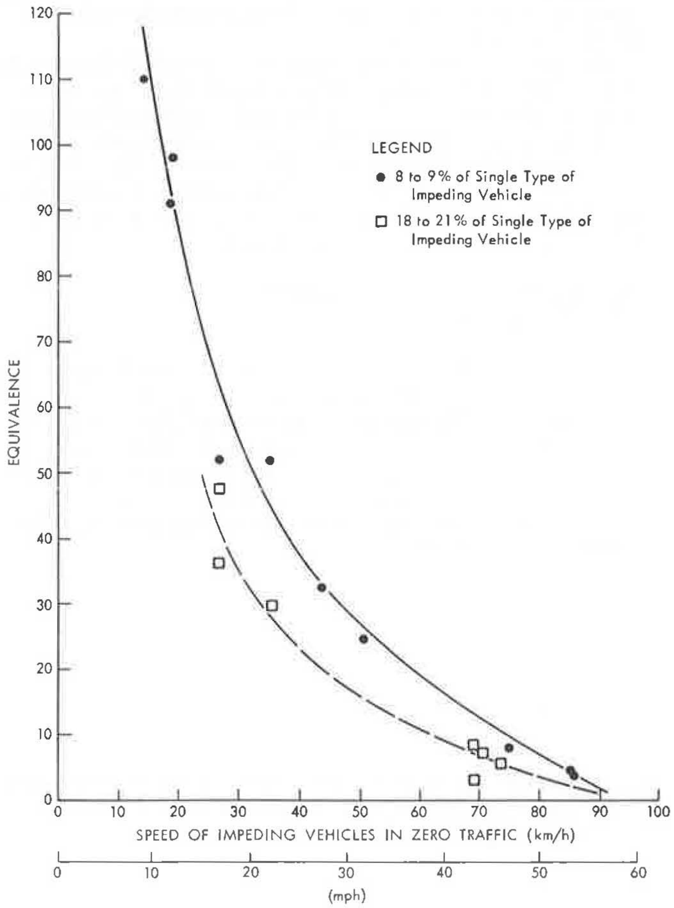
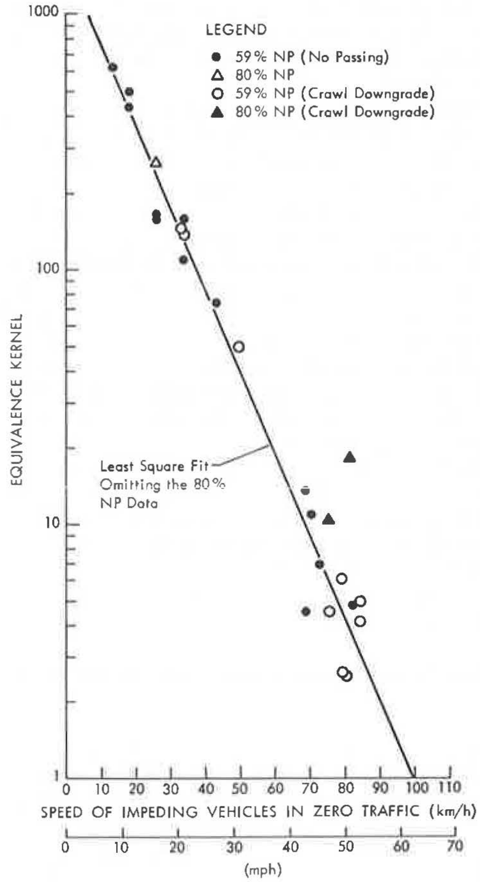


Figure 4. Equivalence kernel versus speed for a 105-km/h (65-mph) speed limit.



$$(Q_E/Q) = (2r + 1)^{1/2} \tag{8}$$

The truck factor becomes $1/(2r + 1)^{1/2}$ and, for the simple case of one impeding vehicle type,

$$r = (P_T/100)(\nu - 1) \tag{9}$$

where ν is defined as the equivalence kernel, a new term that means magnitude depends on the speed of the impeding vehicle type.

In the more general case of n types of impeding vehicles, r is obtained from

$$r = \sum_{i=1}^{n} (P_i/100)(\nu_i - 1) \tag{10}$$

where ν_i is the equivalence kernel for the i th type, which occurs with percentage P_i.

For a flow with only one type of impeding vehicle present at percentage P, the magnitude of the equivalence kernel is obtained from equation 8 as

$$\nu = (50/P)[(Q_E/Q)^2 - 1] + 1 \tag{11}$$

Equation 8 is a fundamental relation. As shown later this equation provides passenger vehicle flow rates that are the equivalents of mixed flows. The mixed flows can contain impeding vehicles in varying quantities and mixes. Equation 10 provides the format to assemble r for a mix of impeding vehicles. Equation 11 provides a format to evaluate ν for an impeding vehicle type when ν is the single impeding vehicle type in the mixed flow Q.

The ν (or the set of ν_i) are not equivalents; they are defined as equivalence kernels. That is, the kernels are assembled and subjected to a nonlinear process in equation 8 before equivalence in the usual sense is quantified.

The simulation results used to construct Figure 3 are used to calculate ν by equation 11. The results are shown in Figure 4. The variance around the least squares fit in Figure 4 does not depend systematically on percentage of impeding vehicles as in Figure 3. The fitted equation for equivalency kernels is

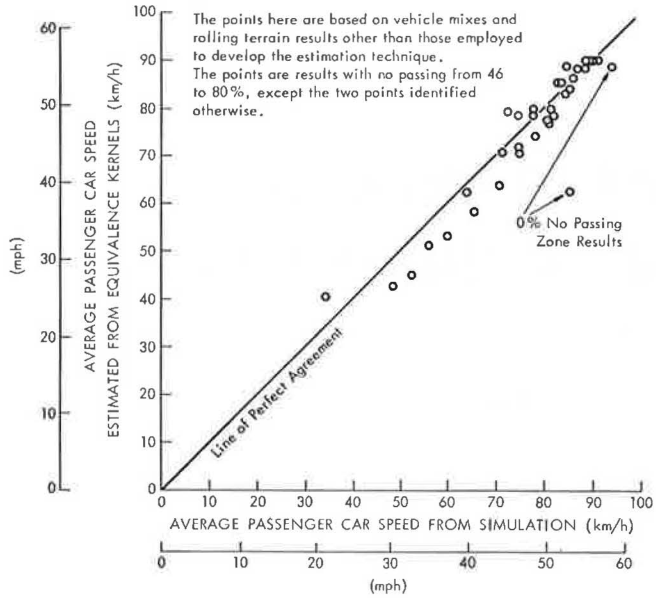
$$\nu = e^{(7.440436 - 0.0749846\,V)} \tag{12}$$

where V = impeding vehicle speed in kilometers per hour. Equation 12 is applicable for flows that are nearly balanced on highways where the percentage of no-passing is 46 to 80 percent and where the 85th percentile speed of passenger cars is about 105 km/h (65 mph) in light free-flowing traffic. The numerics in equation 12 should change for highways with different design speeds or speed limits and for highways with a percentage of no-passing outside the range 46 to 80 percent. With lower design speeds and limits, the intercept ν = 1 should occur at a lower impeding-vehicle speed. With a smaller percentage of no-passing zones, ν should change less rapidly with V.

SUPPORT FOR THE NONLINEAR TRUCK FACTOR

Other results from the simulation have been used to further test the concept of an equivalence kernel and the associated equations. The additional test uses simulation

Figure 5. Comparison of estimated values and simulation results of mean speeds of passenger cars.



results in which there are a mix of impeding vehicle types, rather than a single type, and cases of rolling terrain as well as steady grades. The tests involve the ability to predict passenger car mean speeds (or equivalent passenger car flows) by using equations 10, 8, and 12 (Figure 4, then Figure 2). The procedure for prediction involves the following steps:

1. Estimate the mean speed of each impeding vehicle type over the terrain of interest by using the vehicle performance equations and the mean speed of light free-flowing traffic on sections with good geometrics as the speed desired;
2. Using the mean speed available for each impeding vehicle type, apply equation 12 or Figure 4 to obtain an equivalence kernel, ν_i, for each impeding vehicle type;
3. Apply equation 10 to obtain r;
4. Apply equation 5 to obtain the equivalent flow rate of passenger cars only; and
5. Enter Figure 2 and read passenger car mean speed versus Q_E (use the curve for the grade involved for long, steady upgrades and the curve for the 0 percent grade for long, steady downgrades, 2 percent grades, or rolling terrain).

The values calculated by using the above procedure are compared with simulation results in Figure 5. The agreement indicates that the estimation method provides useful results. However, there is a small systematic deviation that is not associated with grade, grade length, vehicle population, or the choice of 59 percent or 80 percent no-passing. The estimated speeds below 70 km/h (43 mph) are consistently low by 5 to 9 km/h (3 to 6 mph). However, this deviation should be considered in perspective. Similar tests employing the equivalents associated with the linear truck factor currently in use provide very high equivalent flows and correspondingly low estimates of mean speed when mixes of impeding vehicle types are used.

EQUIVALENCE KERNELS FOR USE IN THE NONLINEAR TRUCK FACTOR

Equivalence kernels are given in another report (2) for trucks and recreational vehicles in rolling terrain and on sustained grades. The values available are limited to the highway speeds and percentages of no-passing previously identified. There is a need to extend the results from the simulation model to other highway speeds and to a wider range of no-passing percentages.

ACKNOWLEDGMENT

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Measures of Pedestrian Behavior at Intersections

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This research was performed as part of a research project to identify and evaluate intersection improvements for pedestrian safety at urban intersections. Two field studies and a series of field observations were conducted to identify measures and methods that could reliably yield information concerning potentially hazardous pedestrian behavior at intersections. Both operational measures and conflict measures were investigated. Of 16 behavioral measures that were tested at 120 intersections in the original field study, 7 were retained, refined, and tested in the following field study. These measures showed considerable promise in differentiating the high from the low accident intersection of a matched intersection pair (having similar traffic controls and geometrics). The measures that were developed in this task were to be used in the evaluation phase of the project.

This paper presents the results of a task, which was one of several in a research project (1), to develop and evaluate a set of pedestrian-vehicle measures. Several field studies were conducted to identify measures and methods that could reliably yield information concerning potentially hazardous pedestrian behavior at intersections. The measures investigated were of two general types: operational measures and conflict measures.

Operational measures, such as volume, queue formation, and delay, have long been accepted as useful in establishing the characteristics of pedestrian and vehicle movement. Not nearly so well established, conflict measures have been developed for certain vehicle studies primarily to measure the hazards of traffic zones, such as intersections. Unfortunately, in the specific area of pedestrian countermeasures, conflict measures have not been well established. The relation between such measures and the long-term pedestrian accident history of an intersection has not been demonstrated.

In previous observational studies, a single data collection procedure was usually followed. Mainly, manual observation and hand coding of pedestrian and vehicular activities were used. In some studies the manual tallies of vehicular pedestrian volumes were the major data source; in other studies real-time and time-lapse photography was used to record vehicular and pedestrian behavior. In relatively few studies were pedestrians and drivers interviewed to determine attitudes toward or reasons for their behavior. A notable exception to the reliance on a single procedure and the absence of interview data was the Berger study (2). Consequently, we drew heavily on the methods and findings of that study in developing the measures presented below.

REQUIRED CHARACTERISTICS OF THE BEHAVIORAL MEASURES

To be useful, a behavior had to possess the following five characteristics:

1. The behavior had to be definable in objective, observable events so that coding would be reliable.
2. The behavior had to occur with sufficient frequency to permit an efficient data collection schedule.
3. The behavior had to have an association with intersection safety or operation (assumed or proven).
4. The behavior had to be sensitive to the differences between intersections. Conflict measures had to, in addition, discriminate on the basis of accident history or vehicle-pedestrian flow. Sensitivity was emphasized for several reasons. Validating the measures would provide considerable guidance in the selection and modification of candidate measures. The selected conflict measures could be used by city engineers to determine warrants for intersection treatment. The acceptance of the countermeasures would depend on proof of their effectiveness. Thus, the behavior used to evaluate the countermeasures must be meaningful and believable to the city traffic engineer.
5. The behavior had to be measurable by currently available and cost-effective techniques.

APPROACH TO THE DEVELOPMENT OF BEHAVIORAL MEASURES

The initial step was to establish and collect behavioral measures for a set of intersections for which a complete set of accident records was available. Since two inter-

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sections having the same traffic controls and similar geometrics often have different accident records, we concluded that different operational and conflict levels would be associated with the two intersections. Our intent was to determine which behaviors were most often associated with high pedestrian accident intersections.

We identified a set of high pedestrian accident intersections (where three or more accidents had occurred) in Washington, D.C., San Francisco, and Oakland, California, and matched a number of these intersections with low pedestrian accident intersections (where pedestrian accidents were 50 percent or less than those that occurred at the high accident locations). This matching procedure ensured that the evaluation of measures would be conducted within a common situational context, i.e., intersections with similar attributes. This design avoided confounding the results of our sensitivity study with the physical attributes of the intersections.

The field portion of the task was designed to develop measures that would be reliable, be easily applied, have wide application, discriminate between the intersections, and be related to the intersection pedestrian safety record. In addition, the field studies would provide insight into the variety of operational problems at different intersections and their possible remediation.

FIRST FIELD STUDY

A set of behaviors (Table 1) was generated from the Berger study (2); these behaviors were defined and all measures involved were field tested.

Simultaneously, a listing of the frequency of pedestrian accidents (from 1971 through 1973) by intersection was generated, and all intersections experiencing three or more accidents were designated as potential study sites. Review of the geometric characteristics of these intersections showed that the vast majority had four legs; therefore, only four-legged intersections were considered in the field studies. Sixty of the high accident intersections (45 in Washington, D.C., and 15 in California) were matched with low accident intersections having similar geometrics. In well over 90 percent of the cases, the matched intersections (referred to as a pair) shared a common road and were within several blocks of each other. The following are some characteristics of the 60 pairs:

Characteristic	Number of Pairs
Right angle, two-way, two-way (15 were in California)	30
Skew, two-way, two-way	15
Right angle, two-way, one-way	15
Not signalized	19
Signalized	24
Traffic signalized and pedestrian signalized	17

These 60 pairs of intersections served as the test bed for the development of the behavioral measurement procedures. A subset of these site pairs was used in the selection of promising behavioral measures and identification of potential intersection accident causal characteristics.

Development of Behavioral Measurement Procedures

We decided to attempt to gather the required data by manual tallies and observational procedures because previous studies had proved nonmanual techniques to be costly. For example, Berger (2) found that 3 h were required to reduce a 15.2-m (50-ft) roll of time-lapse film that had required only ½ h to photograph.

A preliminary set of data collection forms was designed for recording the candidate behaviors that had been selected (Table 1). Also, forms were designed for the collection of pedestrian and vehicle volume data.

Approximately 10 people were trained by classroom instruction and by in-the-field practice to use the preliminary forms. During the training sessions, the behavioral definitions were continually refined. Months of effort were devoted to making the measures operative. The measurement procedures were then standardized and the set of data collection forms revised.

A sample of the 60 intersection pairs was selected to determine the reliability of the data collection procedures for each of the measures of interest. The results of this reliability analysis, given in Table 2, indicated that the reliabilities were all high. These results demonstrated the feasibility of using the developed procedures to select the most promising behavior measures.

Selection of Promising Behavioral Measures

The collection of the behavior measure data represented the major effort of the task. Teams of field investigators visited each site to collect the behavioral and operational data. The procedures that were developed indicated that from one to four field investigators would be needed per intersection (depending on pedestrian volumes).

The data collection schedule was designed to sample the morning peak, off-peak, and afternoon peak traffic. A minimum of 3 h of data were collected at each intersection in a pair by field investigators who traveled back and forth between the two intersections. An additional data collection requirement was that at least 100 pedestrian crossings be observed at a pair and that a minimum of 40 crossings be observed at one of the intersections.

A continuous review of the collected data and field notes indicated that some intersection pairs should be discarded. At some sites, construction was started after the arrival of the field team; at others, the signals at one site in a pair became inoperative. In some cases, differences in geometrics became evident. After careful investigation 38 intersection pairs were selected. These intersections had the following characteristics.

Characteristic	Number of Pairs
Right angle, two-way, two-way (eight were in California)	21
Skew, two-way, two-way	6
Right angle, two-way, one-way	11
Not signalized	15
Traffic signals	20
Traffic signals and pedestrian signals	3

Table 3 presents a summary of the data from the 38 selected intersection pairs. The second column in this table gives the percentage of intersection pairs exhibiting 5 percent or more of a particular type of behavior; that is, at least 5 percent of the pedestrians at one of the intersections performed the behavior. Only those intersections where the behavior could occur were included in the calculations. Thus, if an intersection pair did not have signals, it could not have any pedestrians crossing against the signal (CA) and would be excluded from the CA calculations. Fewer than half of the intersection pairs exhibited 5 percent or more of the following behaviors: A, D, RC, VR (pedestrian), VL (pedestrian), SC, VS, VR (vehicle), and VL (vehicle). Because of their infrequent occurrence, these behaviors were eliminated or redefined. The third column of the table indicates the percentage of intersection pairs at which a particular behavior occurred more frequently at the high

Table 1. List of candidate pedestrian behaviors.

Behavior	Symbol	Definition
Abort	A	Returning to curb after placing both feet on the roadway or abandoning crossing to cross intersecting street
Backup movement	B	Momentarily reversing or hesitating after starting to cross roadway because of the threatening approach of a vehicle
Diagonal crossing	D	Crossing intersection diagonally
Running in roadway	R	Running in roadway after having entered roadway
Running into roadway	RC	Running into roadway from curb
Outside crosswalk	OC	Crossing all traffic lanes outside painted crosswalk (not coded for unmarked crosswalks)
Crossing against signal[a]	CA	Crossing all traffic lanes against pedestrian or traffic signal
Starting during caution signal[a]	SC	Starting to cross roadway during caution phase of signal
Starting against signal[a]	SA	Starting to cross roadway against pedestrian or traffic signal although walk or green signal appears before crossing is completed
Straight-through vehicle[b]	VS	Being within 6 m of and in path of nonrestricted vehicle proceeding straight through intersection
Right-turning vehicles	VR	Being within 6 m of and in path of vehicle turning right into the crosswalk
Left-turning vehicles	VL	Being within 6 m of and in path of vehicle turning left into the crosswalk
Vehicles moving through crosswalk, then turning right	VT	Being in conflict with vehicles moving through crosswalk and then turning right
Vehicle overtaking	VO	Entering roadway and moving in front of stopped or pausing vehicle (not a parked vehicle) into lane of traffic moving in same direction
Moving vehicle	MV	Being in traffic lane while vehicle going straight moves through crosswalk
Proximity of vehicle	PV	Entering traffic lane while vehicle approaches six car lengths or fewer away

Note: 1 m = 3.3 ft.
[a] Signalized intersections only. [b] Nonsignalized intersections only.

Table 2. Interrater reliability of sampling of pedestrian behavior.

Behavior	Mean Correlation Coefficient[a]	Number of Independent Pairs of Coders
A	0.9724	5
B	0.8485	7
D	1.0000	3
R	0.8113	6
RC	0.8451	5
OC	0.8599	7
CA	0.8623	7
SC	0.9175	4
SA	0.8872	7
VS	—[b]	—
VR	0.8843	7
VL	0.8816	4
VT	—[b]	—
VO	—[b]	—
MV	0.7508	7
PV	—[b]	—

[a] Each correlation coefficient was based on a sample of 20 cycles. All mean correlation coefficients were statistically significant at the 0.01 level.
[b] Activity occurred too infrequently to calculate a correlation coefficient.

Table 3. Summary of pedestrian behavior recorded at 38 selected intersection pairs.

	Intersection Pairs (%)[a]		
Behavior[b]	≥5% of Pedestrians Exhibited Behavior	Behavior Occurred More Frequently at High Accident Intersection	Percentage of Pedestrians Exhibiting Behavior Was Greater at High Accident Intersection
B	86.8	68.4	52.6
R	86.8	57.9	42.1
RC	44.7	31.6	42.1
OC	54.0	37.8	37.8
MV	100.0	73.7	55.3
VS (pedestrian)[c]	53.3	46.7	60.0
VR (pedestrian)	31.6	44.7	55.3
VL (pedestrian)	21.0	34.2	42.1
PV[c]	93.3	60.0	46.7
CA[d]	82.6	56.5	34.8
SC[d]	35.7	57.1	71.4
SA[d]	91.3	69.6	52.2
VS (vehicle)[c]	0.0	53.3	60.0
VR (vehicle)	42.1	55.3	50.0
VL (vehicle)	28.9	42.1	52.6

[a] Based on the total number of intersection pairs at which the behavior could occur. Intersection pairs at which behavior occurred with equal frequency or did not occur at all were treated as having occurred less frequently at the high accident intersection.
[b] Behaviors identified by (pedestrian) are based on the number of pedestrians involved in that behavior and those identified by (vehicles) or the number of vehicles involved.
[c] Unsignalized intersections only.
[d] Intersections with traffic or pedestrian signals only.

accident intersection. These data indicate the ability of a behavior to differentiate a high accident location from a low accident location. The fourth column of the table indicates the percentage of intersection pairs at which the percentage of the occurrence of a behavior was higher at the high accident intersection.

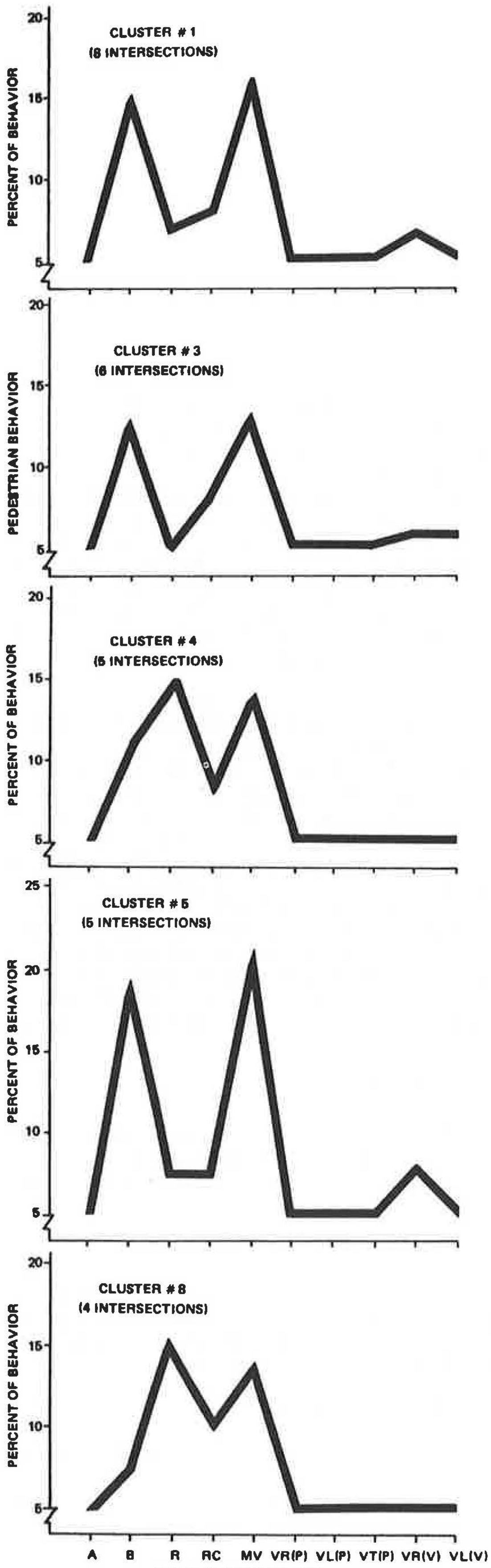
An analysis of these data by the Fisher's distribution free sign test was performed to determine which behaviors significantly differentiated between the high and low members of a pair. This analysis dealt only with the direction of the difference (more frequent at high site equaled a plus, less frequent at low site equaled a minus) and ignored ties. This analysis revealed that the following behaviors occurred more frequently at the high accident sites: B, MV, SA, VR (vehicle), and VL (vehicle).

Although these results appeared to be promising, the high accident sites were noted to have heavier pedestrian volumes. Thus, these differences in frequencies could be attributed to the fact that generally more people were present to exhibit these behaviors. (When we used the percentage of pedestrians exhibiting each behavior as a measure, we found no difference between the percentages at the high and low accident sites.) On the other hand, the differences in the frequency of these behaviors could have been sufficient to contribute to the differences in the accident histories of the intersections.

Based on these results, we decided to further examine the percentage of pedestrians performing each behavior to determine whether a combination of behaviors could be used to differentiate high and low accident intersections. Using a program developed by Yoo, Schmitz, and Berger (3), we classified intersections based on the percentage of pedestrians performing 10 specific behaviors: A, B, R, RC, MV, VR (pedestrian), VL (pedestrian), VS (vehicle), VR (vehicle), and VL (vehicle). These behaviors were selected because they could occur at any of the 38 site pairs. Behavior A, which occurred with extremely low frequency and was therefore omitted from the previous univariate analyses, was included in this analysis since it might interact with other variables. The program compared each intersection with every other intersection. Intersections having similar percentages of pedestrians involved in the same behaviors were clustered together.

Eight clusters of four or more intersections were created; in all, 36 of the 76 intersections were placed into one or more of these clusters. Although seven of the eight clusters contained either all high or all low accident intersections, one cluster contained four low and one high intersections. The success of this classification process was impressive: The program treated each intersection individually and not as a member of a

Figure 1. Profile of low accident intersection clusters.



pair; therefore, signalized and nonsignalized, two-way and one-way, and right angle and skew intersections were all classified by using the same scheme.

The graphs for each of the eight clusters are presented in Figures 1 and 2. A distinctive feature of the low accident clusters (as shown in Figure 1) is that they were bimodal; i.e., the percentage of MV's was equal to the percentage of B's (clusters 1, 3, and 5) or R's (clusters

Figure 2. Profile of high accident intersection clusters.

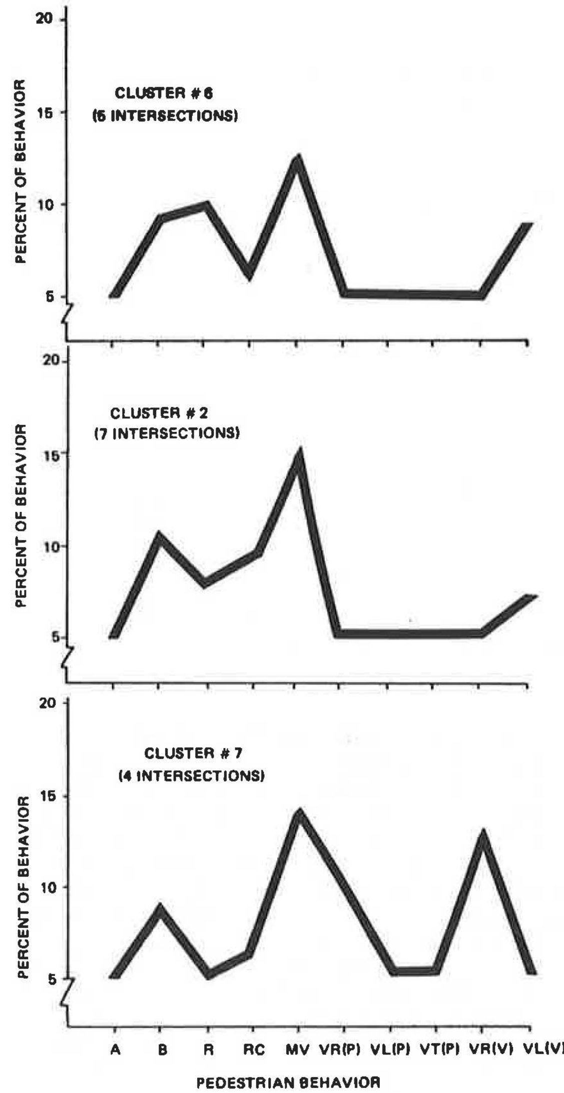


Table 4. Revised list of pedestrian behaviors.

Candidate Behavior	Symbol	Definition
Backup movement	B	Momentary reversing or hesitating after starting to cross roadway because of threatening approach of a vehicle
Moving vehicle	MV	Being in traffic lane while vehicle going straight moves through crosswalk
Turning vehicle	TV	Being within 6 m of and in path of a turning vehicle
Vehicle hazard	VH	Entering a traffic lane while an unrestricted vehicle approaches within one block
Running vehicle hazard	RVH	Running in a traffic lane in response to a vehicle hazard
Running vehicle turning conflict	RTV	Running in a traffic lane in response to a turning vehicle or a potential turning vehicle

Note: 1 m = 3.3 ft.

Table 5. Summary of revised pedestrian behaviors recorded at nine intersections.

	Signalized Intersections[a]												Nonsignalized Intersections[a]					
	1		2		3		4		5		6		7		8		9	
Behavior	Frequency	%	Frequency	%	Frequency	%	Frequency	%	Frequency	%	Frequency	%	Frequency	%	Frequency	%	Frequency	%
B	+	+	+	+	+	+					+	+	+	+			+	+
MV	+		+	+	+	+	+	+			+		+	+	+		+	+
TV[b]	+		+	+							+	+	+	+	+	+		
TV (RTV)[c]	+	+	+	+			+	+	+	+			+		+	+		
TV[d]	+	+	+	+							+		+	+	+	+		
RTV[e]	+		+	+			+	+	+	+			+		+	+		
TV + RTV	+	+	+	+						+	+		+		+	+		
VH	+		+		+	+	+	+			+	+	+	+	+		+	+
RVH	+	+	+	+	+	+	+	+	+	+	+	+	+	+			+	+
VH + RVH	+	+	+		+	+	+	+			+	+	+	+	+		+	+

[a] + indicates that the high accident site exhibited a greater frequency or percent of the stated behavior.
[b] TV percentage = number of TV's per number of pedestrians.
[c] RTV percentage = number of RTV's per number of pedestrians.
[d] TV percentage = number of TV's per number of vehicles.
[e] RTV percentage = number of RTV's per number of vehicles.

Table 6. Pedestrian and vehicle volumes at nine intersections.

Intersection Number	Type[a]	Pedestrians	Vehicles Turning Left	Vehicles Turning Right	Vehicles Going Straight
1	H	1125	124	120	1044
	L	624	216	76	1140
2	H	853	104	192	1168
	L	269	108	132	960
3	H	138	40	40	452
	L	156	20	60	368
4	H	139	16	44	408
	L	130	28	18	700
5	H	149	40	56	1220
	L	130	64	64	1072
6	H	530	100	172	1144
	L	365	64	60	906
7	H	154	24	72	236
	L	76	16	12	144
8	H	123	64	44	304
	L	94	56	48	328
9	H	120	8	8	228
	L	72	8	12	168

[a] H = high accident intersection; L = low accident intersection.

4 and 8). The clusters containing the high accident locations (Figure 2) had a considerably higher MV percentage than B or R. Although the previous analysis indicated that B's occurred more frequently at the high accident sites, this analysis indicated that the percent of pedestrians displaying B behavior was less. Perhaps high accident locations would have a better accident record if a proportional number of people hesitated for vehicles (B) or ran in response to vehicles (R). Also, of the three high accident clusters, only cluster 6 consisted of nonsignalized intersections. This cluster displayed a higher percentage of R's than cluster 2 or 7. This finding was in keeping with accident data analyzed in another task of the project: A higher percentage of R-behavior pedestrians were hit at nonsignalized intersections than at signalized intersections.

The analysis of the clusters indicated that the following five variables most influenced the differentiating between high and low accident locations: B, R, MV, VR (vehicle), and VL (vehicle).

These results tended to confirm earlier analysis and indicated the predictive value of several of the behavioral measures. (R was added, although SA was excluded from this multivariate analysis since SA could only occur at the 46 signalized sites.)

Identification of Possible Intersection Accident Causal Characteristics

The final step in the identification of accident causal factors was to perform a detailed site survey in Washington, D.C., of 30 of the 38 intersection pairs. The previously collected data were used to guide the investigation of each site pair. Additional site factors that might account for the differences in accident experience were explored during the activity.

Each site pair was reviewed for pedestrian and traffic volume, the nature of the abutting property, and the type of vehicle regulations in effect. The high accident site did not differ from the low accident site in presence of schools, playgrounds, parking regulations or observance, turn restrictions, vehicle volume, or turning volumes. The sites in a pair all had a road in common, and, therefore, no differences in vehicle volumes were expected.

Several significant differences were, however, uncovered. First, the pedestrian volumes were significantly higher ($p \leq 0.05$) at the high accident intersections. Also, the high accident sites were significantly more commercial or higher in density than the low accident locations. The high accident sites significantly more often had a liquor store abutting on the intersection. The age of the accident-involved pedestrian and the time of day that the accident occurred did not indicate that alcohol was a problem at these locations. Rather, we suspected that the presence of the liquor stores was a general indication of the socioeconomic environment surrounding the intersection. These neighborhoods often had a higher density than the low accident sites and appeared to be less desirable than their low accident counterparts.

SECOND FIELD STUDY

Because of the promising nature of the behaviors identified in the first field study, we undertook to further refine the data collection methods and the behaviors in a second field study.

Refinement of Behaviors

Based on the field observations and the subsequent results, a review of the promising behaviors was initiated that stressed the importance of pedestrian safety. Three major questions were asked about each behavior.

1. Does the occurrence of this behavior represent a safety hazard?
2. Are there other behaviors that are not being measured that represent distinctly hazardous situations?
3. Can we improve the procedure by which we measure each behavior?

We concluded that most of the behaviors identified as promising in our previous analyses satisfied question 1.

However, behavior R, by itself, did not appear to be a safety hazard; we would consider this behavior when it occurred in combination with other behaviors.

A consideration of question 2 led to a reevaluation of behavior PV at nonsignalized intersections. Behavior PV seemed to cover an important situation; however, its definition was a source of coding error and was considered too stringent. Therefore, behavior VH was proposed (Table 4) which could be used in combination with behavior R.

Considering question 3 led to the combining of the previous behaviors VR and VL into one behavior, TV, which could also be used in combination with behavior R. A combination of questions 1 and 3 also resulted in the refinement of the definitions of B and MV. A set of revised definitions and symbols (Table 4) led to the revision of the data collection forms, which made it possible to collect over twice as many data per day and greatly simplified the analysis process.

Pilot Testing

The newly defined behaviors and the revised data collection procedures were pilot tested at nine pairs of intersections. These intersections were randomly selected from those used in the first field study. Three pairs had pedestrian signals, three pairs had traffic signals only, and three pairs had no signals.

A two-person data collection team collected data at each site for a day. The data collection followed the schedule used in the first field study. The data collection procedures met the criteria of efficiency and minimum retraining.

A summary of the results from this pilot study is given in Tables 5 and 6. On the basis of this small sample, MV, VH, and RVH were found to significantly differentiate ($p \leq 0.05$) the high from the low accident intersections in a pair. This differentiation was based on the frequency of the behavior to be higher at the high accident site. Behavior RVH also separated the high from the low sites on the basis of the percentage of that behavior to occur at each site. On the basis of this pilot study, behaviors B, TV, and RTV did not significantly differentiate among the sites, but, based on their performance during the first field study and the trends from this second study, they did show some promise of doing so.

CONCLUSIONS

The set of behaviors studied appears to address a variety of pedestrian safety problems revealed during a review of the accident data. Behaviors B, VH, RVH focus on the acceptance of small vehicle gaps on the part of pedestrians and the problems of the short time that the pedestrian is visible to the vehicle driver. TV and RTV are a corollary of the turn-merge accident frequently noted at signalized intersections. MV and SA are indications of pedestrian risk taking. In both cases, the pedestrian is in a travel lane exposed to a potential conflict with a vehicle. In behavior SA, the pedestrian anticipates the walk interval (early starter) and possibly presents a target to vehicles speeding to avoid a red stop signal. Behavior MV can occur any time the pedestrian violates the traffic signal or enters the roadway while through vehicles are still moving through the crosswalk area. The two field studies indicate that use of behaviors discussed above shows considerable promise of providing a future tool for differentiating the high and the low accident intersections.

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Bus Priority System Studies Using Instrumented Buses

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Priority treatment for buses on urban roadways has been implemented in various forms in an attempt to reduce bus operating costs and to encourage more commuters to use buses. This paper deals with one aspect of bus priority: the use of instrumented buses to study the operational effectiveness of bus priority schemes. The studies, which were carried out in Miami, Florida, evaluated two bus priority techniques:

1. Local preemption of traffic signals by buses and
2. Preemption of traffic signals plus use of an exclusive, reversible bus lane.

Data from two stages of operation, each associated with a particular bus priority technique, were collected and compared with the conditions that existed prior to the implementation of the system.

DATA COLLECTION AND ANALYSIS

An automated data collection technique was developed for this study. The objective of the technique was to obtain a large amount of data over a series of short sections of roadway at a low-cost. Placed aboard the bus were cassette recorders that were connected to the bus odometer to record the trajectory of the bus in 30-m (100-ft) increments. The raw trajectory information from each sample run was analyzed by computer and combined with corresponding information from several similar runs to generate nine measures of effectiveness:

1. Average speed,
2. Running speed,
3. Total delay,
4. Stopped delay,
5. Travel time,
6. Fuel consumption (estimated from travel parameters),
7. Number of stops,
8. Number of speed changes, and
9. Speed noise.

STUDY RESULTS

The data collected during the three stages of the project (including a preliminary stage) showed that the use of the preemption system resulted in substantial improvements and that the use of an exclusive bus lane further improved the bus operations. The most important measures of effectiveness are summarized in the following table, which gives the percentage of improvement that was measured at each stage during the afternoon peak period, with the signal preemption and with the addition of the exclusive, reversible lane.

Measure of Effectiveness	Signal Preemption	Bus Lane
Travel time	25	8
Total delay	61	15
Speed noise	46	12
Number of stops	68	19
Speed changes	27	12
Estimated fuel consumption	7	3

Several interesting relationships between the measures of effectiveness were also developed. The average speed in a given section was found to be strongly correlated with the delay, number of stops, number of speed changes, speed noise, and fuel consumption. Of the three measures of effectiveness proposed as indicators of passenger comfort, speed noise was suggested as the preferred measure, since the number of stops tended to be somewhat inconsistent in the lower speed range and the number of speed changes tended to diminish at speeds less than 24 km/h (15 mph), giving a false indication of passenger comfort.

The measurement technique shows considerable promise for widespread use. Both the hardware and the software are reasonably simple and contain no proprietary constraints. Efforts are under way to simplify the process so that it can be used by many other agencies and studies.

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